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THE ASSOCIATION was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 800 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Life Member shall be a member whose service to the Association entitles him to special recognition by the Association.

A Junior shall be not less than eighteen years nor more than twenty-five years of age, a student or connected with water supply work.

An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

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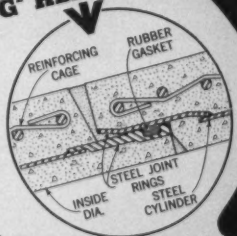

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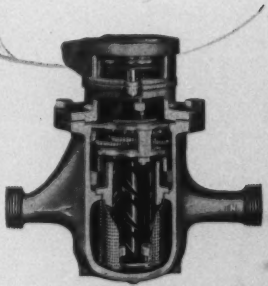

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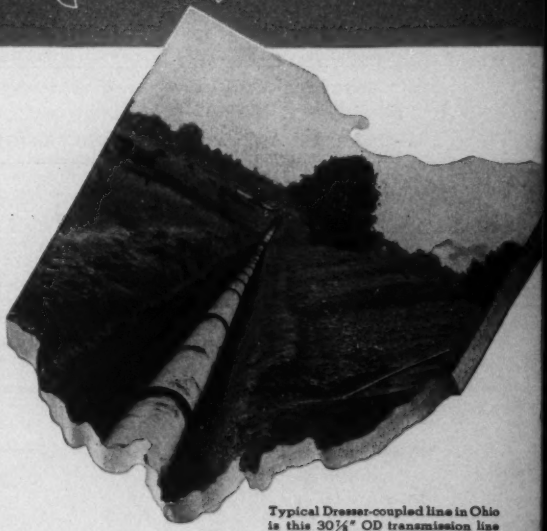

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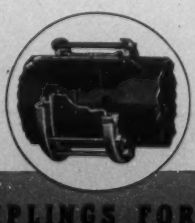

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DESIGN AND CONSTRUCTION OF PRESSURE AQUEDUCT FOR BOSTON METROPOLITAN DISTRICT

BY STANLEY M. DORE*

[*Read December 19, 1940*]

The Massachusetts Metropolitan Water Supply Commission has recently completed its New Pressure Aqueduct.† This aqueduct delivers the waters of the newly finished Quabbin supply from the Ware River and Swift River watersheds into the Metropolitan Water District of Boston in such fashion as to keep the water as free as possible from pollution or danger of it. At the same time it increases the pressures in the district so as to eliminate pumping and reduce the extent of high-pressure service areas. The new aqueduct extends about 18 miles in an easterly direction across the eastern part of the state from an intake in the town of Southborough (about one-half mile easterly of the terminal chamber of the Wachusett Aqueduct) to a downtake shaft near the Charles River in the town of Weston. From this shaft, it will be continued, under subsequent contracts, as a pressure tunnel to Chestnut Hill Reservoir in Brookline. This pressure tunnel will be driven at a depth of several hundred feet below Boston and its suburbs, and will ultimately be extended to encircle the entire Metropolitan District.

The New Pressure Aqueduct has a carrying capacity, under present conditions, of 200 m.g.d. and of 300 m.g.d. under later probable conditions. The present water consumption of the district is about 140 m.g.d. In order to deliver this water under hydrostatic conditions that will eliminate pumping and in order to tie in this long aqueduct advantageously with existing distribution reservoirs, it was necessary, with the limited heads available (about 6 to 10 ft.) to design an aqueduct of very large diameter. Studies of aqueduct locations,

*Assistant Chief Engineer, Metropolitan District Water Supply Commission, Boston, Mass.
†"Pressure Aqueduct for Boston", by Karl R. Kennison, Eng.-News Record, August 31, 1939.

furthermore, determined the advisability of going under the Sudbury Reservoir in Framingham in tunnel.

The Pressure Aqueduct was built in cut-and-cover 150 in. in diameter for 9,680 ft., as pressure tunnel in rock 168 in. in diameter for 15,820 ft., and in cut-and-cover 138 in. in diameter for 67,700 ft., 2300-ft. of cut-and-cover aqueduct, 84-in. in diameter connect with the existing Weston Aqueduct at Sudbury Dam and Weston Terminal Chamber. The 138-in. aqueduct connects to and is constructed across a high-level distribution reservoir in Weston. This section of the aqueduct consists of a twin, reinforced-concrete line 1306 ft. long, with two horseshoe-shaped cross-sections 7 ft. 3 in. high by 9 ft. 2 in. wide.

This paper deals particularly with the design, manufacture, delivery, laying and testing of the cut-and-cover sections of the pressure aqueduct.

Design of Aqueduct

Thorough studies of various types of pressure aqueduct led, as stated in the Joint Board report (House 262 of the 1938 Legislature), to the conclusion that while a tunnel would be the best type of structure for this conduit, it would be a very costly structure in the suburban areas in comparison with cut-and-cover aqueduct; also that for large size of aqueduct required, the most advisable type of cut-and-cover conduit, from an economical standpoint, would be a precast, reinforced-concrete, steel-cylinder, pipe similar to that used in the Colorado River Aqueduct for the Metropolitan Water District of Southern California. Work on that project had demonstrated not only the feasibility but also the economy of cut-and-cover pressure aqueduct.

This type of conduit consists of a line of precast concrete units with joints that are watertight under the pressure and other conditions of service. The precast units are manufactured of reinforced concrete in which are embedded (a) a steel cylinder, for absolute watertightness under pressure, and (b) two cages of reinforcing steel. The concrete protects the steel against deterioration. Figure 1 shows a cross-section of the conduit. On the inside of the pipe, two inches of unreinforced concrete cover the steel cylinder. Three major factors must be taken into consideration in the design of this pipe: (1) the internal working pressure , (2) external loads on the empty structure,

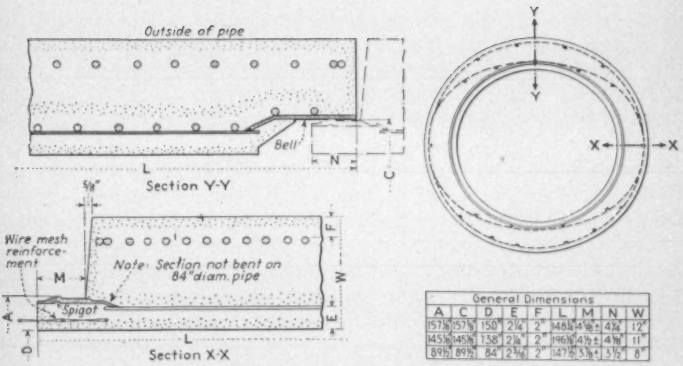

FIG. 1.—CROSS-SECTION OF PIPE AND JOINT DETAILS.

and (3) the dead load of the pipe itself. The last two factors influence the stiffness required and affect the choice of wall thickness. This is 12 in. for the 150-in., 11 in. for the 138-in. and 5 in. for the 84-in. pipe. Handling the pipe units during casting, curing and delivery operations are also considered in determining wall thickness, and the proper value is largely a matter of empirical knowledge rather than of theoretical computation.

The internal pressures allowed for varied from 100 to 220 ft. of water, no pipe being designed for less than a 100 ft. head, even though in some instances the working pressure was less than this amount. The cut-and-cover cross-section called for a cover of 30 in. of earth over the top of the pipe. No special allowance had to be made for this, however, because other design factors made the pipe so strong that it could carry not only this load but a surcharge of from 10 to 25 ft. of earth, depending on the amount of reinforcing. This permits construction of highways along or across the aqueduct without requiring an additional structural envelope.

The steel cylinders were designed to take 40 to 50% of the stress induced by internal pressure, the remainder being carried by the reinforcing cages. The allowable stresses were purposely set low, about 10,000 lb. per sq. in. in order to prevent excessive cracking of

TABLE 1—DETAILS OF PIPE REINFORCEMENT.

Pipe Diam., In.	Design Head, Ft.	Total Cross Sectional Area of Steel Per Ft. of Pipe Wall, Sq. In.	Cylinder		Circular Cage			Elliptical Cage		
			Thickness, U.S. Std. Gage or In.	Cross Sectional Area Per Ft. of Pipe Wall, Sq. In.	Rod, Diam., In.	Spacing In.	Cross Sectional Area Per Ft. of Pipe Wall, Sq. In.	Rod, Diam., In.	Spacing In.	Cross Sectional Area Per Ft. of Pipe Wall, Sq. In.
150	100	4.02	#10	1.66	11/16	3.774	1.18	11/16	3.774	1.18
138	100	3.70	#10	1.66	5/8	3.609	1.02	5/8	3.609	1.02
	110	4.07	#8	2.02	5/8	3.591	1.025	5/8	3.591	1.025
	120	4.45	#8	2.02	11/16	3.666	1.215	11/16	3.666	1.215
	130	4.81	#6	2.39	11/16	3.681	1.21	11/16	3.681	1.21
	140	5.19	¼	3.00	5/8	3.346	1.10	5/8	3.346	1.10
	150	5.56	¼	3.00	11/16	3.480	1.28	11/16	3.480	1.28
	160	5.93	¼	3.00	47/64	3.458	1.47	47/64	3.458	1.47
	170	6.30	9/32	3.38	47/64	3.481	1.46	47/64	3.481	1.46
	180	6.66	¼	3.00	7/8	3.943	1.83	7/8	3.943	1.83
	190	7.04	¼	3.00	7/8	3.572	2.02	7/8	3.572	2.02
	200	7.40	¼	3.00	1	4.284	2.20	1	4.284	2.20
	210	7.78	¼	3.00	1	3.943	2.39	1	3.943	2.39
	220	8.14	9/32	3.38	1	3.960	2.38	1	3.960	2.38
84	100	2.29	#12	1.29	7/16	3.61	0.50	7/16	3.61	0.50
	120	2.76	#10	1.66	7/16	3.28	0.55	7/16	3.28	0.55
	140	3.21	#10	1.66	17/32	3.43	0.775	17/32	3.43	0.775
	150	3.44	#6	2.39	7/16	3.44	0.525	7/16	3.44	0.525
	160	3.67	#8	2.02	17/32	3.22	0.825	17/32	3.22	0.825
	170	3.90	#8	2.02	5/8	3.91	0.940	5/8	3.91	0.940
	180	4.13	#6	2.39	5/8	4.23	0.87	5/8	4.23	0.87
	190	4.36	#6	2.39	5/8	3.74	0.985	5/8	3.74	0.985
	200	4.58	#6	2.39	5/8	3.36	1.095	5/8	3.36	1.095
	210	4.82	#6	2.39	11/16	3.66	1.215	11/16	3.66	1.215
	220	5.04	#6	2.39	11/16	3.36	1.325	11/16	3.36	1.325
	140	3.21	#10	1.66	1/2	3.04	0.775	1/2	3.04	0.775

150-in. Pipe: Longitudinal rods 26@¼ in. by 1 in.=6.5 sq. in.; bell joint rings 5/16 in. by 10½ in. weighing 11.16 lb. per ft.
138-in. Pipe: Longitudinal rods 24@¼ in. by 1 in.=6.0 sq. in.; bell joint rings 5/16 in. by 10 in. weighing 10.63 lb. per ft.
84-in. Pipe: Longitudinal rods 20@½ in. round=3.93 sq. in.; bell joint rings ¼ in. by 7½ in. weighing 6.38 lb. per ft.
All spigot joint rings weighed 7.96 lb. per ft.

the concrete under pressure. This means that the allowable stress in the cylinder will be not more than 20,000 to 25,000 lb. per sq. in. even though no assistance is obtained from the cages. This insures watertightness of the aqueduct even under some unexpected condition of deterioration of the concrete, or cages, and accident to the concrete envelope.

Table 1 shows the steel required in the precast concrete units for various working heads. In computing the resistance to internal pressure, both the steel cylinder and the reinforcing cages were assumed to act together with uniform stresses. In the pipes of large diameter, the reinforcing cages were wound so that two cages were formed, one a circular cage concentric with the pipe axis and the other an elliptically-shaped cage of the same horizontal diameter as the circular cage (which was generally about 4 in. smaller than the finished outside diameter of the pipe) but of shorter vertical diameter so that the elliptical cage rested on the steel cylinder. The use of cages of this shape permits a distribution of the reinforcing steel to resist more adequately the moments in the pipe shell induced by external loads, with concentrations at the outside of the shell at the horizontal diameter and at the inside of the shell at the vertical diameter.

The concrete is not considered to aid in resisting internal pressures, but its degree of strength will affect the wall stiffness for resistance against external and dead load stresses. The concrete acts primarily as a protection of the steel against deterioration and other damage. Hence, it was deemed advisable to require a dense impervious abrasive-resistant concrete of fairly rich proportion, which would then be a high-strength concrete as well. In this work, the quantity of cement used was paid for separately in order that the engineer could control its use to best advantage.

The pipe joints were made of steel joint rings into which could be caulked a lead gasket for watertightness. The general features of the joint required (1) that when the pipes were laid they would be self-centering without the aid of the gasket, (2) that non-uniform settlement of adjoining pipes would not increase the bearing load on the gasket, and (3) that the rings should be accurately formed so that, when adjoining pipes were put together, a satisfactory sliding fit for the self-centering surfaces with a tolerance of not more than 1/32 in. from a close sliding fit would be obtained. Figure 1 shows a detail of

the type of joint used. A ridge cast on the spigot prevents the gasket from being deformed by uneven settlement. The bell ring consists of a plate of rectangular cross-section, bent as indicated, and the spigot ring has a specially rolled cross-section of the shape indicated. These bell and spigot rings were formed accurately and evenly to their proper diameter and carefully checked on a templet for size and roundness before they were welded to the steel cylinders. The gasket is a fiber-filled, extruded, lead gasket of wedge shape, so made and joined that, for each joint, the gasket is endless and of proper circumference to fit loosely between the spigot ring and the bell ring of the joint. Reinforcing mesh was cast in the spigot end of each pipe to bond the mortar filling of the caulking recess to the pipe spigot, thus insuring proper backing of the lead gasket at all times. The manufacturer was required to build the pipes in lengths not greater than 16 ft. He elected to make the 150-in. and 84-in. diameter pipe 12 ft. long, the 138-in. pipe 16 ft. long.

The center line of the aqueduct was located with due regard to real-estate damages and construction features. Soundings were made along the aqueduct line to determine the presence of bedrock. Where bedrock might be encountered, core borings were made to determine the exact position of the ledge surface. The aqueduct was located to best advantage, not necessarily in a straight line in plan, and more or less followed the contour of the ground, uphill and downdale to avoid, so far as practicable, costly excavations and fills. Figure 2 shows typical sections of the aqueduct in earth and rock excavation and on embankment. The pipe was laid so that it would carry adequately the stresses due to external loads. To this end, compacted refill was placed around the pipe in addition to the construction of a cheap, lean concrete cradle that was poured around the lower portion of the pipe for a width approximately equal to the internal diameter of the conduit. The purpose of this construction was materially to strengthen the resistance of the pipe walls to external loadings by creating a wide bearing across the base and compacted refill for support of the sides.

The pipe was fabricated, cast, cured and delivered by the manufacturer, and a general contractor excavated the trench, laid the pipe, made and caulked the joints, poured the cradle and finished the refill around it. The contract for the manufacture of the pipe was let before the contracts for laying it. In order that the pipes could be manu-

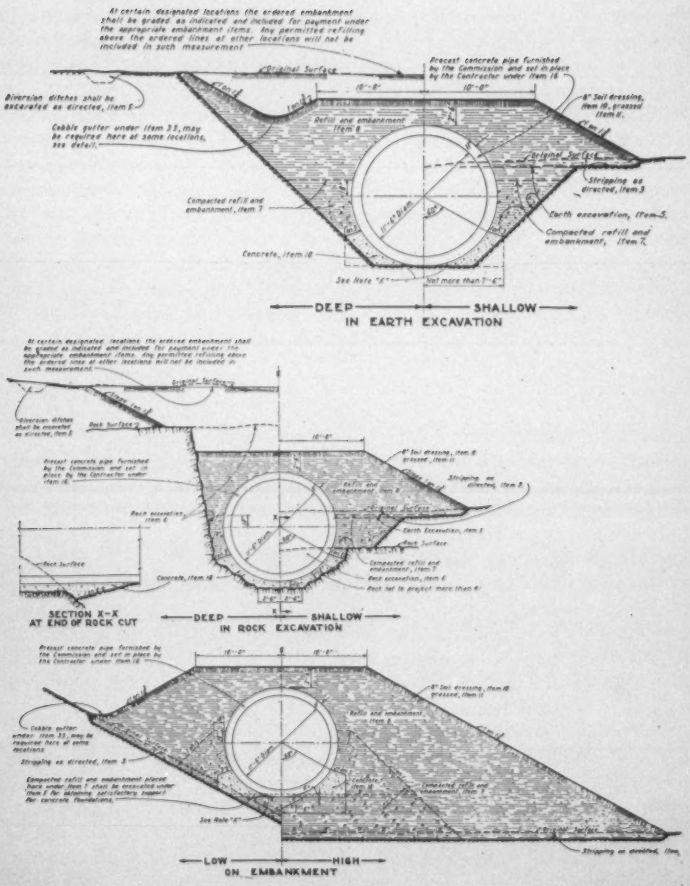
At certain designated locations the ordered embankment shall be graded as indicated and included for payment under the appropriate embankment items. Any permitted refilling above the ordered lines at other locations will not be included in such measurement.
Precast concrete pipe furnished by the Commission and set in place by the Contractor under Item 16
Original Surface
Diversion ditches shall be excavated as directed, Item 5.
Refill and embankment Item 6
6" Soil dressing, Item 18, grassed Item 6.
Cobble gutter under Item 33, may be required here at some locations, see detail.
Original Surface
Stripping as directed, Item 3
Compacted refill and embankment, Item 7
Earth excavation, Item 5.
Compacted refill and embankment, Item 7.
Concrete, Item 18
See Note "E"
Not more than 1'-6"
DEEP
SHALLOW
IN EARTH EXCAVATION
Rock Surface
Rock excavation, Item 4
Earth Excavation, Item 5
Compacted refill and embankment, Item 7
Rock not to project more than 6"
SECTION X-X AT END OF ROCK CUT
DEEP
SHALLOW
IN ROCK EXCAVATION
Cobble gutter under Item 33, may be required here at some locations
Stripping as directed, Item 3
Compacted refill and embankment placed here under Item 7 shall be excavated under Item 5 for obtaining satisfactory support for concrete foundations.
See Note "E"
Stripping as directed, Item
LOW
HIGH
ON EMBANKMENT

FIG. 2.—TYPICAL SECTIONS OF AQUEDUCT SHOWING EXCAVATIONS AND EMBANKMENTS.

factured by one contractor and laid by another in a predetermined location, it was found advisable to identify in advance the position of each joint along the line of aqueduct. In this way, pipes containing connections or forming curves could be cast of proper dimensions in advance. All curves were created by the use of bevel-ended pipe. The angle of the bevel was approximately 4½ degrees in all cases. This means that curves are made up of a series of cords or tangents about 16 ft. long. With proper advance computations, the vertical and horizontal curves were negotiated in all cases, without the use of special lengths of pipe at the ends of curves except that in a few instances the manufacturer cast half-lengths of pipe. The necessity for laying out these joints in advance may be appreciated also from the fact that the pipe was designed for changes in hydrostatic heads of 10 ft. This meant also that the position of joints at hilly locations was needed in order that the design of each section of pipe could be determined before the unit was cast. The joint was constructed so that a leeway of about 1 in. horizontally was available in its making, allowing a gain or stretch of ½ inch in laying lengths. With this tolerance, the laying contractor had little difficulty in placing the joints at the locations predetermined from a precise survey. The bevelled ends were manufactured so that the spigot end of a straight adjoining pipe would fit into the bell directly parallel to the center line without any cramming or difficulty. This was effected by welding the bell ring to the steel plate in a plane perpendicular to the axis of the adjoining pipe regardless of the position of the axis of the pipe to which the bell was welded. Each pipe was marked at the yard with the manufacturer's number and with a laying number. The laying number was used in assigning the delivery and the laying locations in the field and avoided the installation of pipe at incorrect locations.

The economical depth of laying the pipe was usually about 10 ft. in earth. Where the ground water level was more than 7 ft. above the invert of the pipe, it was necessary to weight the line against floating when empty and uncovered. This was done by increasing the size of the cradles and providing anchoring the pipe to the cradle. Steel reinforcing anchors were screwed into inserts that were welded to the pipe cylinder and extended through the pipe wall. The inserts were manufactured with the pipe, and threaded reinforcing hooked rods were inserted in them after the pipe had been laid in the trench. Figure 3 illustrates the anchorage.

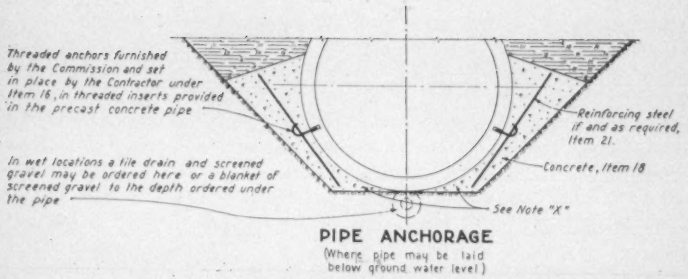
Threaded anchors furnished by the Commission and set in place by the Contractor under Item 18, in threaded inserts provided in the precast concrete pipe
In wet locations a tile drain and screened gravel may be ordered here or a blanket of screened gravel to the depth ordered under the pipe
Reinforcing steel if and as required, Item 21.
Concrete, Item 18
See Note "X"
PIPE ANCHORAGE
(Where pipe may be laid below ground water level)

FIG. 3.—PIPE ANCHORS.

In regions of considerable ground water, the design called for the use of crushed stone and tile pipe to care for water during construction. Longitudinal drains were plugged with concrete after the aqueduct was laid. The aqueduct crossed several swampy areas. These were stiffened by the use of crushed stone so that adequate support would be furnished for the heavy pipe units. All peat and muck were removed from the pipe foundations, but quicksand under the peat was satisfactorily handled by the use of heavy gravel or crushed stone.

The backfill and embankments of the aqueduct were dressed with 8 to 12 in. of loam and grassed. The side slopes of 1 on 13/4 appear to be wearing well where proper drainage is provided. At highway crossings, the pipe was strong enough to carry the highway loads under a cover of about 3 ft. of earth. At most highway crossings, highway grades were improved after construction, there being little difficulty in keeping the grade of the aqueduct low enough. Wherever possible, surface drainage was carried over and across the aqueduct. As a rule, the top of the refill over the aqueduct was situated above the original ground surface. This necessitated carrying surface drainage parallel to center line of the aqueduct in longitudinal ditches (paved where steep enough so to require) and thence under the aqueduct or away from it. Ordinarily inverted siphon culverts were needed. These consisted usually of rectangular sections with a minimum cross-section of waterway of 2 ft. by 3 ft. Each culvert was built with flared ends, as shown in Figure 4. This increased carrying capacity and reduced velocities of approach and discharge.

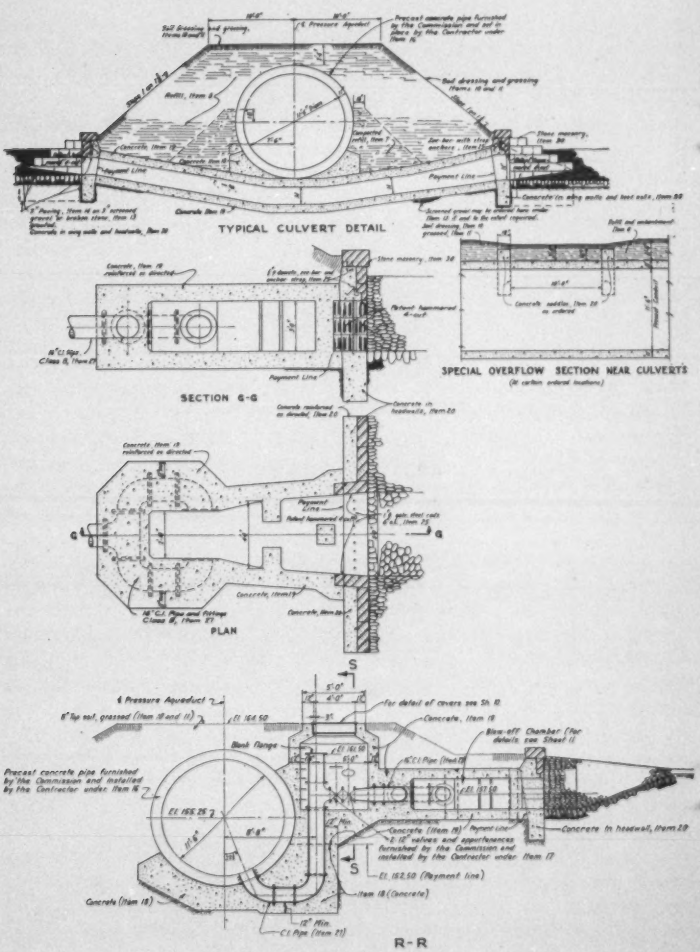

FIG. 4.—TYPICAL DETAILS OF CULVERT AND BLOW-OFF.

Wherever the embankment covering the aqueduct rose above the adjacent ground for any considerable distance, a depressed spillway section was built over the pipe which would come into use in case culverts should clog and create upstream pondage of water. The purpose of this emergency spillway was to localize and minimize damage to the embankment. Two concrete collars were cast around the aqueduct at the spillway site so that erosion would be confined to the fill over the pipe between the concrete collars and water would thereupon flow over the aqueduct as over a weir between these collars, see Figure 4.

The aqueduct is provided with access manholes about 400 ft. apart. All sections of the aqueduct are equipped with blowoffs and air valves. All sections, therefore, can be thoroughly drained off at low points in the line, although, in some instances, pumping is required to obtain complete unwatering at river crossings. The blow-off piping is designed to dissipate the energy of the discharged water in two jets that impinge upon one another as indicated in Figure 4. The summits of the line are equipped with automatic vacuum and pressure valves which let air in and out of the line. At changes in grade, which were not necessarily peaks, manually operated air valves are placed, in order to release pockets of air. The manually operated air valve is a small bronze unit of 1½-in. nominal size and of special design. It is screwed into an 8-in. connection on the pipe that is so arranged that the water in the aqueduct will tend to keep this fitting from freezing; see Figure 5.

As the pipe has very strong walls, there was no necessity for designing it against collapse under vacuum. As a rule, the air valves for this large line consist of nominally 8-in. units, each of which is a cluster of four 4-in. air-intake valves and one pressure-relief valve. The largest installation consists of two such 8-in. installations. The access manholes are combined wherever possible with blow-off or air-valve structures. A typical design of both blow-off and air-valve manholes is shown in Figures 4 and 6.

Plate-steel bulkheads were used for testing completed sections of the aqueduct. The contract required that each laying contractor test his section under full hydrostatic head for leakage, and that this leakage should be less than 75 g.p.d. per inch diameter, per mile of pipe. Figure 7 shows a typical detail of the bulkhead.*

*"Plugs for Aqueduct Testing" by Stanley M. Dore, *Eng.-News Record*, February 29, 1940.

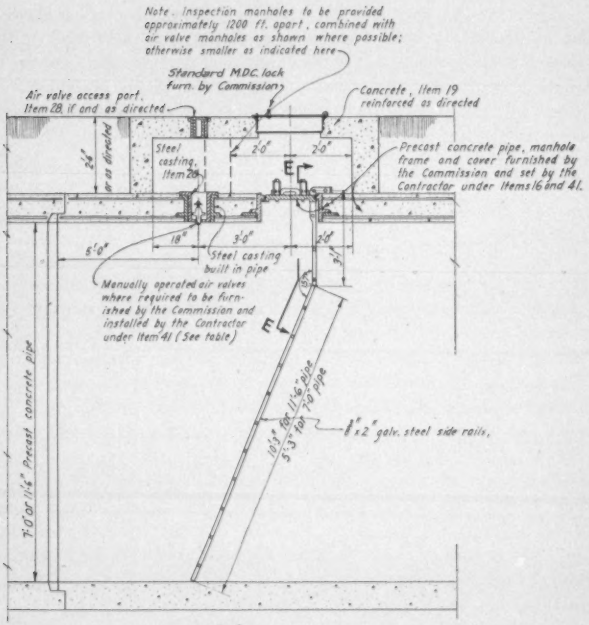
Note. Inspection manholes to be provided approximately 1200 ft. apart, combined with air valve manholes as shown where possible; otherwise smaller as indicated here
Standard M.D.C. lock furn. by Commission
Concrete, Item 19 reinforced as directed
Air valve access port, Item 28, if and as directed
Steel casting, Item 28
2'-6" or as directed
2'6"
2'0"
Precast concrete pipe, manhole frame and cover furnished by the Commission and set by the Contractor under Items 16 and 41.
6'0"
18"
3'0"
2'6"
Steel casting built in pipe
Manually operated air valves where required to be furnished by the Commission and installed by the Contractor under Item 41 (See table)
7'0" or 11'6" Precast concrete pipe
10'3" for 11'6" pipe
5'3" for 7'0" pipe
8" x 2" galv. steel side rails,
C-C

FIG. 5.—MANUAL AIR VALVE INSTALLATION.

Provisions for future connections to the aqueduct were made at nearly all main highway crossings; and if the time of connection seemed not too far distant, a valve was installed in order that the aqueduct would not have to be shut down to make the connection. Elsewhere, a flanged outlet capped with a blank flange was employed. Reinforced concrete manholes for access to these connections were built. Connections to the pipe, whether for blow-offs, air valves, future connections, or manholes, were made of steel castings welded to the steel cylinder of the pipe.

There are main-line valves in the large aqueduct at two points,

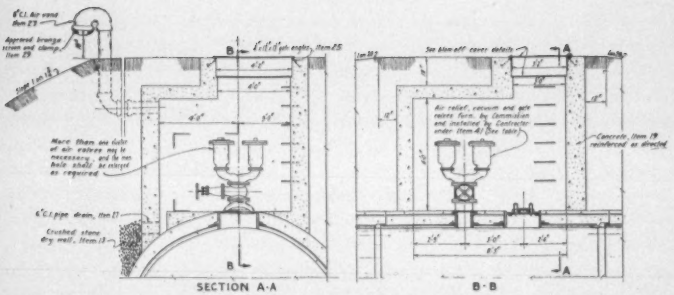

FIG. 6.—AIR VALVE MANHOLES.

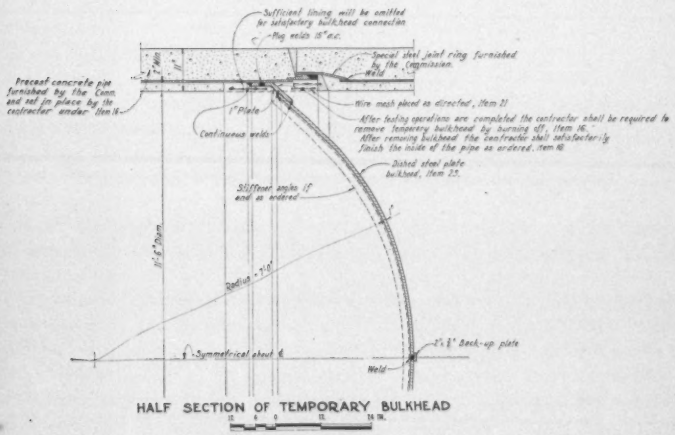
Sufficient lining will be omitted for satisfactory bulkhead connection
Special steel joint ring furnished by the Commission
Weld
Precast concrete pipe furnished by the Comm. and set in place by the contractor under Item 14
1" Plate
Continuous weld
Wire mesh placed as directed, Item 21
After testing operations are completed the contractor shall be required to remove temporary bulkhead by burning off, Item 16. After removing bulkhead the contractor shall satisfactorily finish the inside of the pipe as ordered, Item 18
Dished steel plate bulkhead, Item 25.
Stiffener angles if and as ordered
Radius = 7'-0"
Symmetrical about ℄
Back-up plate
HALF SECTION OF TEMPORARY BULKHEAD

FIG. 7.—TYPICAL BULKHEAD FOR TESTING.

one near the Sudbury Dam in Southboro and one near the 84-in. connection to Weston Reservoir in Weston. These main line valves consist either of 60-in. Dow valves or 60-in. gate valves. The aqueduct is reduced in diameter from 138 in. to 108 in. by an eccentric reducer. Connection is then made to two 60-in. valves by an eccentric

Y called a "pair of pants". The invert grades of the aqueduct were kept in line so that drainage is not affected. The transitions are constructed of welded steel plate, reinforced with structural-steel shapes welded to the plates where stiffness is required, the steel structure being encased in reinforced concrete and lined with mortar. The valves were joined to the transitions by cast-steel adapters that furnish a rigid anchored connection to the pipe on one side of the valve and a slip joint on the other. This joint makes it possible to remove the valve and takes up play due to expansion or contraction or other minor movements when the valve is put under pressure.

Anchorage at curves was simplified by limiting all changes in direction to 4½ degrees at any joint. No complicated designs were needed, but special care was exercised during construction to obtain thorough compaction of the backfill at all curves.

In the entire 15 miles of pipe, there are no expansion or contraction joints, except at main-line valves, and the aqueduct is forced to adjust itself to temperature changes by increasing or decreasing temperature stresses in the concrete sidewall. A rise in temperature of 10° F., for example, increases the compressive stresses in the concrete walls by about 200 lb. per sq. in.

Where the aqueduct crossed highways, the only utilities appreciably affected were local water lines. In some towns, these were relocated by constructing inverted siphons under the aqueduct at the crossings when the cover over the aqueduct was not deemed adequate for frost protection. The aqueduct was laid across several larger brooks which flowed over the completed pipe practically unchanged after construction. One of the river crossings was that of the Sudbury, with a drainage area of about 100 square miles at the point of crossing. The pipes were anchored to rock in the river bed, and covered with earth protected by stone in the river bottom and at the shores.

Manufacture of Pipe

A contract for furnishing and delivering the precast concrete pipe was executed with the Lock Joint Pipe Company, Ampere, New Jersey, on December 28, 1938, the lowest of four bidders. The bid price was $4,164,126.50, and the value of the final estimate, dated November 2, 1940, was $4,198,409.03. The manufacturer built his main plant on Speen Street, Natick, near the Boston-Worcester turn-

pike, about half-way between the ends of the 138-in. aqueduct. At this main plant all the 150-in., 138-in. and 84-in. steel cylinders and cages were fabricated and concrete plants were set up here for casting the 138-in. and 84-in. pipe. Another concrete plant was built in the town of Southboro, much nearer to the location of the 150-in. line. To this plant were hauled the 150-in. cylinders and cages, and here were cast the 150-in. concrete pipes, thus avoiding transportation to the upper end of the line of the large and heavy 150-in. units.

Erection of the Natick plant started early in January, 1939, and required about three months. During this time, the site was graded; steel forms for casting the pipe were fabricated; base rings were cast in concrete for pouring the pipe; water and steam lines were laid, and standard-gage spur railroad tracks were built. A large steel plant was erected in addition to the concrete plants. This plant was a building, about 70 ft. by 300 ft., in plan, in which the steel cylinders were welded, rolled and tested hydraulically and in which the reinforcing cages were fabricated. The concreting plant included a cement storage bin, a mixing plant with two 2-cubic yard mixers, a dinky locomotive and flat cars for hauling the concrete to the casting site, a locomotive crane for depositing the concrete and special rotary hoppers on the tamping platforms, and vibrators used in putting the concrete into the forms.

The contract required delivery of the pipe units to start by June 1, 1939, and to be completed before April 1, 1940. The contractor started delivery approximately on schedule, and before the end of 1939 he had manufactured practically all of the pipe and had delivered about 90% of it, making remarkable progress in its manufacture. Fabrication of the steel cylinder and cage assemblies for both the 150-in. and 138-in. pipes was sub-let to the American Concrete and Steel Pipe Company of Los Angeles, California, which had built similar large units for the Metropolitan Water District of Southern California and had available some machinery and equipment suitable for this job. The steel plates for the cylinders and the plates and special sections for the bell and spigot rings were delivered by railroad freight near the easterly end of the steel plant. The rings were brought to a ring shop where they were rolled roughly to shape and the ends welded together. They were then stretched over a mandrel beyond the elastic limit to the accurate diameter required, removed from the mandrel, and tested

for roundness and size on a templet or gage. Stretching the rings beyond the elastic limit automatically tested the strength of the weld. The steel plates for the cylinders were sheared accurately to size and then butt-welded to each other. Steel shells were made up of welded sheets about 4 ft. wide and 12 or 16 ft. long, varying in thickness from 10 gage to 9/32 in. Nine or ten of these sheets were employed to form the cylinder. A special automatic welding device which traveled at a desired constant rate of speed along the joints produced a uniformly heated full weld in the joint. The plates were then lifted by a crane and placed around a cylindrical form of proper diameter. The final shell joint was then lap-welded by hand, and the cylinder removed from the form. After this, the bell and spigot rings were hand-welded into the top and bottom of the cylnder. Welding the joint and spigot rings took place in a special templet so that the shape of the joint rings was not distorted during rewelding. The shell was then submitted to a hydrostatic test at a pressure that created stresses of 23,000 lb. per sq. in. in the steel plates. To do this the cylinder was placed over a solid bulkhead tank about 4 in. smaller in diameter than the cylinder so that water could be introduced into the annular space between the inner bulkhead and the cylinder. The lower end of the testing machine contained a water-tight head into which the bell of the cylinder fitted. A second testing head was placed over the top of the shell during the test. Any defect was rewelded and the cylinder again subjected to test.

The reinforcing bars for the cages were delivered in coils, and each coil was put on a spool and fed mechanically to a cage-winding device that consisted of a mandrel rotated by a motor. The shell after being tested was placed on the mandrel and held there by expanded segments. When the mandrel was rotated, the reinforcing rod passed from the machine and was wound to definite spacing over the mandrel. Spacer bars were clamped to the cylinder to give an elliptical shape to the inner reinforcement gage which was in contact with the pipe cylinder at top and bottom and held several inches from the cylinder at the horizontal axis. The elliptical cage was wound on first and followed by the cylindrical cage placed midway between the elliptical coils. See Figure 8. After completion of both cages, the entire steel assembly was removed from the mandrel by an overhead crane and the assembly delivered to the casting yard.

In the casting yard, at the Natick plant, there were 72 bases and

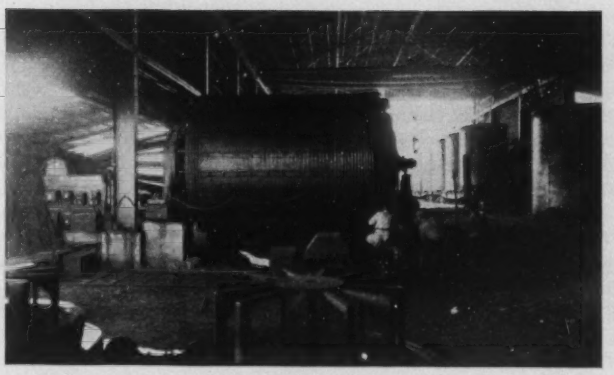

FIG. 8.—WINDING OF CAGES.

24 sets of steel forms for casting the pipe. Each base consisted of a carefully machined, adjustable, cast-iron ring embedded in a concrete foundation. The ring contained grooves into which the inside and outside forms of the steel pipe fitted. All pipes were cast on end, and the manufacturer's routine consisted of pouring 24 pipes per day. The forms were stripped in 24 hours and the new concrete covered with a canvas and steam-cured. After 48 hours, the pipe was turned on its side, strutted and rolled into a storage yard for 12 days' curing under water jets.

The cement used was manufactured to the specifications developed by the late Thaddeus Merriman and adopted by the Water Supply Commission for its work on the pressure aqueduct and elsewhere. A complete description of the tests and properties of this cement may be found in a paper by Thaddeus Merriman published in the *Journal of Boston Society of Civil Engineers* in January, 1939. Cement which fulfills these specifications was found to be of excellent quality insuring, when properly used, a very tough, dense, high-strength, and impervious concrete. If properly mixed and cured, this concrete seems to include very little, if any, free lime and to possess a very high percentage of what Mr. Merriman called amorphous constituents, that is, particles which are not crystalline in structure and which are in-

soluble in water. Concrete made with this cement is particularly resistant to abrasion, chipping or gouging. For use in pipe which has to be hauled, rolled, and bumped, there is no known better Portland cement. The concrete produced is particularly difficult to work with tools, so that care in the casting of each pipe unit was required in order that no chipping at the bell or spigot ends would be needed. Chipping of concrete made with cement that meets the Merriman specification proved to be extremely difficult, even when the concrete was green.

The aggregates used in the concrete were obtained from a local deposit of gravel in the town of Framingham, about a mile and a half from the plant. Preliminary investigations of this deposit by test pits indicated that there might be an excess of soft, decomposed stone. To find out whether there were alkalis or other deleterious substances in the soft rock that would affect the chemical behavior of the cement, preliminary tests were made with briquettes formed from a mortar made of this soft rock by grinding the rock sample and mixing the grindings with cement. There proved to be no such harmful substances. The deeper cuts in the gravel pit showed smaller numbers of soft-rock particles than did the shallower cuts. Abrasion and wear tests of the coarse aggregates were otherwise very satisfactory. When the stone was passed through the crusher, the softer rocks broke down into sand.

The contractor erected a plant for sieving and washing the aggregates. The coarse aggregates, consisting of screened gravel and crushed stone, obtained by crushing the larger gravel screened to a maximum size of 1 in. and a minimum size that would not pass a ¼-in. mesh screen. The sand, after washing, proved to be a rather coarse, uniform sand, and standard briquette and color tests showed it to be of very good quality. The grading of both the coarse and fine aggregates was required to meet the "maximum and minimum grading curves" of the Massachusetts Department of Public Works. The sizes of the fine aggregates were, in addition, such that all would pass a ¼-in. mesh screen and not more than 15% would be retained on a No. 4 sieve. After screening and washing, the aggregates were stacked in piles. When needed they were loaded on trucks and transported to the concrete mixing plant in Natick.

Computations indicated that a mixture of fine and coarse aggregates, consisting of about 41% of sand and 51% of stone, would give

maximum density, and that an increase in the proportion of coarse aggregates might be more economical but not so dense. Test cylinders corroborated these computations. A design compressive strength of 5,000 lb. per sq. in. was established, and little difficulty was encountered in meeting this strength with the proportions of aggregates used for density and imperviousness. Tests of compressive strength of all concrete averaged about 6,000 lb. per sq. in. A water-cement ratio of about 5 gal. per sack of cement was used at first. As the work progressed, the water-cement ratio was increased to 5.8 gal. per sack of cement without appreciable drop in the compressive strength. The cement factor at the begining of the work was 2 bbl. of cement per cubic yard. Several tests were made to determine the effect of using smaller quantities of cement. These indicated that as little as 1.65 bbl. per cu. yd. could be used without appreciable effect on the density. The loss in compressive strength was not a factor as it was still about 5,800 lb. per sq. in. for the leaner mix. The appearance and finish seemed to be about the same, although tests made for abrasion and wear indicated that the leaner mix was not so resistant. During cold weather, the amount of cement used was not less than 1.75 bbl. per cu. yd., although under ideal conditions in warm weather just as good concrete could be secured with a value of 1.65.

As the concrete is designed primarily as a protection for the steel, imperviousness and resistance to spalling and decomposition are the qualities considered most important in design. For this reason, the amount of cement used was purposely required not to be too low. The manufacturer objected to the use of less than 2.0 bbl. per cu. yd., but observations and tests made during the progress of the work indicated that reductions below this figure did not result in appreciable loss in the desired qualities of the concrete. Such reductions, considering the large volumes of concrete involved, effected material savings in cement. It is believed that further reduction beyond 1.65 would have been inadvisable because of probable material effect upon the required qualities of the concrete.

Two 2-yd. mechanical mixers, of the rotating-drum type, mixed the concrete at the Natick plant and dumped it into buckets on flat cars that were hauled to the casting yard. The cement was delivered to the mixer from a storage bin which had the capacity of one day's work. The cement was received in bulk and unloaded mechanically

into the bin. The aggregates were trucked from the gravel plant at Framingham as needed and dumped into a hopper below ground from which they were conveyed by a bucket elevator to bins above the mixer.

The cement and aggregates were proportioned by weight in an aggremeter that fed each mixer. An electric Spangler metering valve measured the desired amount of water and recorded the total amount used in a day's work. Inspectors of the Commission had absolute control of the quantities of all materials, including the water, that were used in any mix. Several times each day they ascertained the moisture content of the aggregates and from prepared tables determined the amounts of water to be added. There was, therefore, an accurate control of the water-cement ratio at all times. In addition, the consistency of the mix was checked by visual inspection by an experienced man in case minor adjustments were needed for variations that might occur in any particular batch due to lack of uniformity in the characteristics of the aggregates. The period of mixing required was at least two minutes.

At the casting yard the buckets were lifted from the flat cars by a locomotive crane and the concrete dumped into a rotating hopper on top of the pipe forms. All pipes were cast vertically. The inner and outer steel forms were erected by the locomotive crane, grooves in the cast-iron bases providing annular spaces into which the inner and outer steel forms could be fitted. A tamping platform was placed on top of the pipe forms and held them concentrically in required positions. The rotating hopper was placed on top of the tamping platform, and, as it rotated, it distributed the concrete around the circumference of the platform into the forms in a uniform manner. Electrically-operated vibrators were mounted on the outside of the exterior forms. Some experimentation was carried on with internal vibrators, but their use was not successful because of the height of the thin walls. Long-bladed spades proved to be more efficient in the removal of air and water pockets from the inner forms. See Figure 9.

The first pipes cast showed an excellent finish on the outside of the pipe, but contained a large number of small pock marks on the inside. By adjusting the rate of rotation of the hopper to control the rate of deposition of the concrete in the forms and by the energetic use of the long-bladed spades near the inside form, improved results

FIG. 9.—POURING CONCRETE.

were obtained. Towards the end of the work, the finish on the inside of the pipe was nearly as good as that on the outside, and the pits or pock marks were materially reduced in number and size. Considerable difficulty was experienced with the finish on the inside of the pipe near the bell end because of the loss of mortar at points where the steel forms were not watertight against the base ring; but by the use of proper caulking materials, this difficulty was overcome.

After the concrete was placed, the top of the pipe was covered, and steam at a temperature of 100° to 120° F. was introduced inside the pipe. The forms were removed 24 hours after casting and the "green" concrete pipe was covered with a canvas hood. Steam at the same temperature was admitted beneath the hood for two more days. Some difficulty was experienced in keeping the pipe walls wet, as there was a tendency for the team to dry out the pipe by heating it without wetting it, the steam condensing on the canvas hood and running down without wetting the pipe. By keeping the temperature of the steam below 120° F. and by tying the canvas securely, so that it was more or less in contact with the concrete at all points, the walls were kept wet, and the difficulty was corrected. However, the most promising way to secure the best concrete is to cure the pipe after removal of the forms with warm water instead of steam.

After being cast three days, the pipes were rolled on their sides into a curing yard, where they were cured for twelve more days by keeping them moist inside and outside. Except in cold weather, this curing consisted of sprinkling the pipes with water through a sprinkler system. Sprinkler heads were installed above the pipe outside and on the inside of the pipe. See Figure 10. At temperatures below 45° F.,

FIG. 10.—STORAGE YARD CURING IN COLD WEATHER.

the contractor was required to cover the pipes with canvas and to bleed steam into the inside of the canvas coverings, creating a moist, warm fog around the pipe walls. Too much emphasis cannot be placed on proper curing of the pipe as it affects the final quality of the concrete. The conditions of curing can easily be detected even after the pipes have been delivered in the field. An inspector can, with but little difficulty, determine visually whether or not the curing produced the best concrete. By chipping a small amount of concrete from the spigot end of the pipe, he can tell by the appearance and odor of the break whether the curing was properly done. By the use of a magnifying glass, he can see the amount of amorphous constituents. Even without a glass he can tell, from the general color, whether or not the amount of free lime is excessive and whether or not the amount of amorphous constituents is low. By smell, finally he can determine the relative

degree of free lime present. The presence of free lime and a low percentage of the amorphous constituents indicates a less durable concrete, because constituents soluble in water are undesirable. Concrete high in soluble compounds will lose them when wet and will deteriorate more rapidly on that account.

A testing laboratory was established at the Natick plant for curing and breaking mortar briquettes and concrete cylinders. Three sets of concrete cylinders were made for every 100 cubic yards of concrete poured. One cylinder was broken at the end of seven days' curing and two at the end of twenty-eight days. Towards the end of the work, the seven-day cylinders were omitted except when the nominal mix was changed. The machine for testing the cylinders had a capacity of 7,000 lb. per sq. in. on a 6-in. diameter cylinder, or a total of 200,000 lb. It was manually operated by a hydraulic pump. Samples of the concrete for the cylinders were taken from the chute of the rotating hopper as the concrete was pouring into the pipe forms, care being taken to get a representative sample. The moulds used for the cylinders were generally of steel, although some cardboard moulds were used. There was no apparent difference in the character of the cylinder from either type of mould.

The concrete laboratory conducted two series of experiments, one on water-curing *versus* no efforts at curing, and the other on water-curing *versus* steam-curing. The first investigation revealed that the compressive strength of concrete cured in the hot sun with little or no water sprinkling had a compressive strength only 50% of that of the concrete cured by sprinkling for fourteen days. The second investigation showed that concrete cured with steam had a higher earlier compressive strength, but that the ultimate compressive strength was the same whether it was cured by steam or water; and that, when curing is done by steam the temperature in the concrete should be kept below 120° F. or drying out and cracking will take place. These results indicate that where high early strength is required, curing by use of hot water will produce even better concrete than curing by steam, and will do so with more certainty and less risk of damage to the concrete.

The contract required that whenever the large pipe units, which weighed from 45 to 50 tons, were laid on their side either for storage or for rolling, they should be equipped with radial struts to maintain the

pipe truly circular. The manufacturer provided struts of 2-in. pipes fitted with shoes and an adjustable screw jack on one end. As these struts were rather expensive, he supplemented them with wooden struts and wedges for use when the pipe remained in storage for any considerable length of time after delivery. Three sets of three struts were generally used in each pipe length, one set near each end and one in the middle. See Figure 11.

FIG. 11.—STRUTS, LOADING AND TRAILERS.

In order to determine the strength of welding and the condition of stresses in the reinforced-concrete pressure-pipe, one 12 ft. length of 84-in. pipe was tested to destruction. At destruction, the concrete had cracked in a great many places uniformly around the pipe, and bursting finally occurred at one of the welded joints at 4½ times the designed load. This test demonstrated (1) that concrete even in tension will stress uniformly; (2) that cracking, with stresses in the steel both below and above the elastic limit, is mostly longitudinal and well distributed around the pipe circumference; and (3) that the stresses are distributed more or less uniformly between the cage reinforcement and the steel pipe cylinder as the design anticipated, otherwise, the bursting of the cylinder would not have occurred at the

anticipated loading. One interesting feature of the test was that the interior lining did not stretch with the remainder of the pipe wall but separated from the steel cylinder and cracked at one place sufficiently to transmit the hydrostatic pressure through the crack onto the steel-cylinder wall. Another interesting observation was that no weeping or minor leakage occurred before the complete rupture. Figure 12 shows a photograph of the pipe unit taken just after destruction.

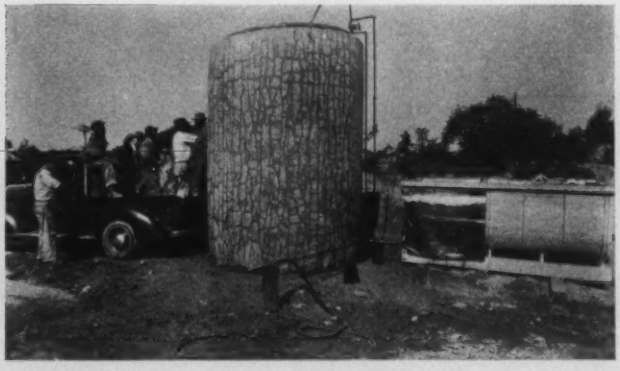

FIG. 12.—PIPE UNIT TESTED TO DESTRUCTION.

STORAGE AND DELIVERY

The manufacturer leveled a large storage area north of the row of casting bases. A specially designed Gantry crane picked up each pipe and turned it on its side to be rolled to the storage area after strutting. The pipe was stored and cured in rows perpendicular to the line of casting bases. The storage yard held about 312 pipes or thirteen days' manufacture and was about two feet higher than a road running along its north edge. Pipes were loaded from this area onto trailers by caterpillar tractors equipped with winches that manipulated steel cables which were wrapped around the pipe in such a manner that the pipe could be rolled from the yard into position on a trailer. Specially designed 50-ton low-bed trailers were pulled by 50 hp. Diesel-powered truck tractors. The trailers had 26 wheels which were

pneumatically tired so that the loading on the road surface was well distributed and approximately equivalent to the ordinary loading from a light delivery truck. Five trailer units were used, each unit making four to six trips a day over an average haul of about seven miles. A caterpillar tractor hauled the pipe from the trailer by a cable. Timbers were used for snubbing operations, for blocking, and for protection of concrete pipes already delivered. See Figure 13.

FIG. 13.—UNLOADING PIPE.

The Commission provided, under the laying contracts, for the building of rough delivery roads along the aqueduct line so that each pipe could be transported and unloaded in a position close to and opposite its final position in the trench. On steep side hills or at brooks, storage areas were leveled off to permit the storage of the pipe units required on the side hill or above the wet area.

Damage to the pipe during delivery was required to be repaired by the manufacturer. On infrequent occasions, slight damage, such as the nicking of the spigot or bell, occurred and was repaired at the point of delivery. As a rule, an earth cradle was excavated along the delivery row in order to receive the pipe unloaded and prevent its rolling after delivery. Stones or timbers were used sometimes to block pipes on level ground. Pipes stored in the open during hot summer weather

would get quite warm, and in many cases crazy cracking of the mortar lining on the interior of the pipe occurred to some extent, although this had no harmful effect upon the soundness of the lining. Examinations revealed that these cracks healed and disappeared after the pipe had been wetted for an appreciable length of time. It is recommended, however, that pipe left in the sun in warm weather should be whitewashed, or coated with white paint to reduce absorption of heat during the middle of the day.

Pipe Laying

In order that the contracts for laying the reinforced concrete pipe might be of a size to be bid upon by smaller contractors, the open-cut aqueduct work was divided into five sections, numbered Sections 1, 3, 4, 5 and 6. Section 2 of the aqueduct being the Southboro Tunnel.

The contract for Section 1 for laying 9680 ft. of 150-in. pipe was executed with B. A. Gardetto, Inc., on March 13, 1939, at a bid price of $503,871.38. Work on this section was substantially completed on August 15, 1940, the value of the final estimate being $521,806.66. This section of aqueduct was laid on a uniform grade for its entire length. Because of this, the contractor was able to use a different type of laying equipment than was required on the other contracts. He

Fig. 14.—Locomotive Crane Laying Pipe on Section 1.

purchased and refitted an old steam-powered locomotive railroad crane and dug a ramp for transporting the crane into position in the open-cut trench. Figure 14 shows this crane in operation. It moved on rails set on ties, laid on the trench bottom. As it was not strong enough to reach the bank, pick up the pipes, and swing them into position in the trench, the contractor dug ramps at right angles to the pipe line and rolled each pipe unit down the ramp, controlling movement with a cable attached to a caterpillar tractor at the top of the ramp. Pipes were delivered, by special arrangement with the manufacturer, at storage areas near the ramps and were rolled to the ramps by pushing them with a bulldozer that was equipped with wooden protective pieces on the blade to prevent damage to the pipes. After the pipe had been rolled down the ramp and onto a special flat car ahead of the railroad crane, the crane pushed the flat car and pipe along the track to the laying position. The crane then lifted the pipe off the flat car and set it into position in the line. Ramps were located about 1,000 ft. apart. The contract included two highway crossings, eight culverts, one emergency overflow spillway, an intake overflow dam and intake works, and steel connections, mortar lined and encased in reinforced concrete, for joining the precast pipe with the intake works and with Shaft 1 of the Southboro Tunnel and for providing future connections at the intake works and at Shaft 1.

The contract for Section 3 for laying 16,524 ft. of 138-in. pipe was executed with B. A. Gardetto, Inc., on May 3, 1939, at a bid price of $497,382.48. Work on this construction was substantially completed on September 15, 1940, the value of the final estimate being $496,597.43. As the grade was not uniform on Section 3, the Contractor elected to use a special Gantry crane or traveling stiff-leg derrick similar to one used by the American Concrete & Steel Pipe Company on Section 4, after experience with it on similar work in California. Figure 15 shows the steam-powered Gantry crane laying a pipe. The back end of this derrick was counterweighted with 200 tons of pig lead to enable the crane to reach out and pick up the pipe at the side of the trench. Pipes could be laid either with the crane traveling forward or backward. In forward motion, the crane had to travel over the top of the completed aqueduct. Less clearance was required at some locations if the unit traveled backwards. The crane seemed to work equally well in either direction. It traveled on two

FIG. 15.—GANTRY CRANE LAYING PIPE ON SECTION 3.

tracks on either side of the trench, a span of 54 ft. was provided on the Gardetto unit. In spite of this long span, there were places, especially in deep cuts, where, unless extra excavation was made, the banks would crumble considerably and even give way during the placing of the pipe. The Gantry unit was clumsy in use on steep grades, as it was heavy and required special anchorage and handling. In swamps and low places, the embankments had to be built in advance of construction in order to provide support for the tracks. A sling was attached to the pipe stored near the edge of the trench, and the Gantry swung its boom over and lifted the pipe, swinging and lowering it into its position in the trench. The work also included a railroad crossing requiring special design and construction features, five highway crossings, ten culverts, and six emergency overflow sections.

Bids on Sections 4 and 5 were opened with the option of awarding either contract separately to different bidders or jointly to one bidder. The low bid in any combination submitted was that of the American Concrete & Steel Pipe Company of Los Angeles for doing the work on both sections. The contract for Sections 4 and 5 for laying 17,834 ft. of 138-in pipe on Section 4 and 18,003 ft. on Section 5 was executed

on March 20, 1939. Work was completed on Section 4 (Contract 78) on August 12, 1940, and on Section 5 (Contract 79) on August 7, 1940. The final cost of the work on Contract 78 was $551,629.18, and on Contract 79 $522,335.25. The company used laying equipment similar to that used for Section 3, except that the span was only 48 ft. This made considerable difference in the amount of earth work necessary, especially in deep cuts and in the fills over swamps. The work involved the construction of the crossing of the Sudbury River, eleven highway crossings, twenty culverts, two blow-off structures, and three emergency overflow sections.

The contract for Section 6 for laying 15,333 ft. of 138-in. and 2,325 ft. of 84-in. pipe was executed with M. F. Gaddis, Inc., on April 26, 1939, at a bid price of $560,432.25. Work on this construction was practically completed in December, 1940, the value of the final estimate being about $600,000. The Contractor on Section 6 elected to use two Speed cranes to handle the pipe, one of 62-ton capacity and the other of 50-ton capacity. Neither crane, however, could lift and swing a 45-ton pipe alone. The larger of the two cranes could merely lift the pipe unit when its boom was in a vertical position. Figure 16 shows the two Speed cranes handling the pipe during laying operations. The cranes worked together in handlng the unit, and although opera-

FIG. 16.—SPEED CRANE LAYING PIPE ON SECTION 6.

tions seemed to be more hazardous, the contractor completed the work with these cranes without serious difficulty. The 84-in. pipe was laid mostly by the larger of the two cranes. The cranes picked up the 138-in. pipe unit and carried it to the edge of the trench and lowered it into the trench. While the pipe was being lowered into the trench, the cranes exerted heavy pressures on the top of the trench. Where the banks were not of good, heavy gravel or stable sand, sloughing was likely to take place underneath the crane caterpillars. On this account the contractor was forced in one location, to use sheeted trench. As a rule, therefore, this method of laying may prove impracticable unless soils are good gravels and the bottom of the trench is such that steep trench slopes are fairly stable. The work on Section 6 involved the construction of ten highway crossings, eight culverts, two blow-off structures, and two emergency overflow sections.

The rates at which the various rigs laid the pipes are as follows: In Section 1, the average rate of laying was about 8.2 pipes per day and the maximum 20 in a 13-hour day. In Section 3, the average rate of laying was about 9.7 pipes per day, and the maximum 31 in a 13-hour day. In Sections 4 and 5, the average rate of laying was about 17 pipes per day and the maximum 32 in a 13-hour day. In Section 6, the average rate of laying 138-in. and 84-in. pipe was about 7.3 and 4.5 pipes per day respectively; the maximum number of 138-in. and 84-in. pipes laid in any day was respectively 27 and 14 in an 8-hour day.

The pipe was usually laid by inserting the spigot end of the pipe being laid to the bell of pipe last laid. The reverse operation was tried on occasion with no appreciable difficulty, although on the whole, laying operations were a little easier when the spigot was inserted into the bell. In such case, the bell of the pipe in place was swabbed with graphite grease and the lead gasket hammered into place in the bell. Then the spigot was entered into the bell, and by swinging the free end of the pipe from side to side and raising or lowering it, the spigot was worked past the lead gasket into its true position, leaving an annular space on the inside of the pipe about 4½ inches wide for caulking. If difficulty was experienced in entering the spigot the required distance into the bell, the pipe was jacked home by hydraulic jacks anchored back into the pipe line in place. The lead gasket was then given an initial caulking. Caulking was done successfully, both by hand and with pneumatic hammers, and it is questionable which is

the better method. Hand caulking is probably a little less wasteful of the lead gasket and has some advantages, although the pneumatic work is satisfactory, and in some instances more thorough. The exterior portion of the joint is usually an annular space varying in width from 1 in. on some pipes to 2 or 3 in. on others. The width depends on the height to which the shoulder of the pipe was cast near the spigot end. The exterior joint was filled with a mortar composed of one part cement and two parts sand. It was forced into position by the fingers of the cement finisher, aided in some cases with a wooden wedge. The backfill was then placed and the interior of the joint was not completed until after the entire line had been laid and settlement of the pipe under its backfield loading had taken place.

Final caulking was done either pneumatically or by hand, and the interior joint was made by filling the annular caulking recess full of mortar, by slinging by hand or by Guniting. The pneumatic process is perfectly satisfactory, but requires skilled operators in order that the sand rebound will not be trapped in any part of the caulking recess. Considerable skill and care are needed to obtain a finish on the Gunited joint equal in smoothness to that obtained on the hand-slung joint. Hand-slinging requires making large balls of Portland cement mortar and throwing them into the caulking recess. In pipe of large diameter, the flatness of the arch is such that complete filling of the joint with mortar in this manner caused the mortar to sag and drop out unless it was supported by forms. Forms were used in Section 3 to support the filling above the springing line of the arch. In Section 4, the difficulty was overcome by placing the filling above the arch in two steps, a foundation filling being introduced in one operation and a finishing course in another. In placing the first filling the mortar was squeezed into the corners where it was retained without sagging. The weight of the second course was not sufficient to cause it to sag if the first course was placed with sufficient corrugations and roughness and had obtained its initial set. Corrugations were roughly formed by squeezing the fingers into the wet mortar during the placing of the first filling. Since the pipes were not truly concentric, there was a slight offset at the joints. Table 2 gives a survey of a number of joints showing the degree of matching.

Several methods were used to lay pipe on steep grades. At the Sudbury River crossing, one contractor handled pipes at the top of the

TABLE 2—OFFSETS OBSERVED IN A SURVEY OF 100 JOINTS.

Offset In Feet	Number of Joints Showing Stated Offset		
		Springing Line of Arch	
	Invert	North Sidewall	South Sidewall
.06 to .07	0	0	4
.05 to .059	0	2	3
.04 to .049	0	0	3
.03 to .039	0	4	13
.025 to .029	4	1	12
.02 to .0249	12	8	15
.015 to .019	21	10	16
.01 to .0149	25	20	9
.005 to .009	18	25	13
Less than .005	20	29	11
Total	100	99*	99**
Average offset—ft.	.0114	.0112	.0219

*Also 1 of .08 ft. on bevel.
**Also 1 of .06 ft. on bevel.

hill with a Gantry crane and slid them down hill into position over two rails laid to proper line and grade in the trench. The rails were greased, and two steel channel sections were placed between the concrete pipe and the rail to aid sliding. Figure 17 shows a pipe being slid into position. The laying in this case would take place uphill.

FIG. 17.—SUDBURY RIVER CROSSING—PIPE SLIDING INTO PLACE.

Another contractor successfully laid pipe downhill on a steep grade with a Gantry crane by anchoring each pipe on the slope from the inside by means of a cable fastened to the bell of the pipe being laid and carried back to a dead man erected in the aqueduct several hundred feet back of the top of the hill. Each pipe was tied separately by a cable to the dead man until all of the pipes had been laid to and beyond the bottom of the hill and until the concreting of all joints had been completed. The cables were then removed and the anchorage and backfill completed. The difficulties involved in downhill construction seemed to be greater than those in uphill construction as the cables had to be tightened and held taut with turnbuckles, and as the holding and manipulation of Gantry crane and pipe on the steep slope were both difficult and risky.

The contractor laid pipe across the Sudbury River at a point where the tributary drainage area is about 100 sq. miles. His work was facilitated by an exceedingly dry season and small flows. The river was diverted into a channel prepared in the valley bottom to the east of the old bed, and the pipe was laid across the westerly half of the old river bed first. The flow was then returned to the original channel, somewhat confined at this time by an earth cofferdam at its easterly edge, and the easterly half of the river crossing was completed. As rock was encountered in the excavations below the old river bottom, an excellent foundation for the pipe crossing was obtained. The pipe was anchored with concrete against flotation. No difficulty should be encountered from flood flows as the pipe is also well protected by several feet of fill and by rock fills on the river bottom and shores that prevent erosion.

The contractors experienced some difficulty in swamps in which peat and muck were encountered in depths of 25 ft. and over. Excavation in these locations demonstrated that the muck and peat were more or less fluid and would flow continuously into the excavation from the limits of the swamp outside. This difficulty was overcome by placing gravel fills on either side of the excavation to be made in the peat. As the excavation progressed, these fills sunk bodily into the peat bog and formed dams that kept the peat from flowing into the excavations and so permitted excavation to be carried to stable materials. At one location, the contractor excavated peat in advance of the gravel foundation refill, getting support for an excavator by depositing gravel ahead of it as excavation progressed.

Where the ground water was so high as to endanger the trench by flooding, the contractor established pumping stations. In a few locations, the bottom was sandy and conditions such as to cause quicksand. Here the contractor drove well points to lower the water table below the trench bottom. He also used crushed stone to stiffen the sands before the pipes were laid. As work had to be done in freezing weather, in order to complete the contract by the required date, the contractors had to provide steam or other forms of heat to thaw out the trench bottom ahead of laying operations and the placing of the concrete cradle. Canvases were usually stretched on the trench bottom and live steam introduced below them through pipes.

The concrete for pipe cradles was usually transit-mixed and purchased from local dealers. The cement used and the mixing of the concrete were supervised at the subcontractor's plant by inspectors of the Commission.

Several closure pieces were needed, at least one on each section of aqueduct. They were of special design, as shown in Figure 18, the

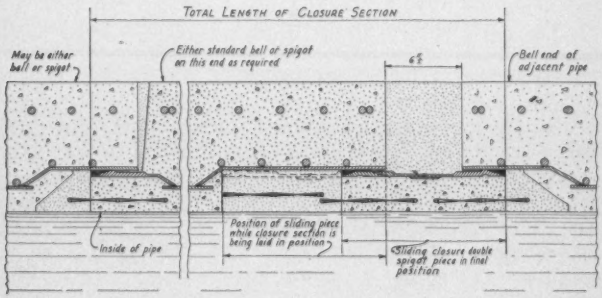
Total Length of Closure Section
May be either bell or spigot
Either standard bell or spigot on this end as required
6"
Bell end of adjacent pipe
Inside of pipe
Position of sliding piece while closure section is being laid in position
Sliding closure double spigot piece in final position

Fig. 18.—Closure Piece—Diagram.

principal feature being a movable double spigot section that could be slid into position in the bell of the pipe at which closure was being effected (after the closure pipe was in place in the line). The closure piece could be caulked from the inside at each spigot end of the movable section to complete the watertightness of the line. The closure piece was cast about six inches shorter than the measured

dimension and could be manufactured some time in advance of use because the pipes were laid fairly accurately to required stationing. The mortar filling of exterior and interior recesses and joints was more extensive at the closures but could be carried on in a manner similar to that adopted for regular joints.

Upon completion of each section of aqueduct it was tested for watertightness under a pressure slightly in excess of the designed working head. The contract required that leakage should not exceed 75 gal. per 24-hr. day per mile of pipe per inch of internal diameter after the pipe had been filled with water for at least 48 hours. These specifications amount to 10,350 g.p.d. per mile of 138-in. pipe and 11,250 g.p.d. per mile of 150-in. pipe. The 48-hour filling period permitted the concrete to become well watersoaked and to swell into its working condition.

Because of the relatively high pressures under which the aqueduct sections were tested, and because of the large diameter of the aqueduct itself, specially designed bulkheads had to be used between sections and at the ends, except at Shaft 4 where valves were available in the line. Each bulkhead consisted of a steel-plate head, welded at its periphery to the steel cylinder of the precast pipe. Both sides of the head were welded at its contact with the cylinder. The head was dome-shaped and designed for pressures from either side, putting the dome shell into tension or compression. Further details of these heads are described in *Engineering News-Record,* February 29, 1940.

Tests on the various sections indicated leakage well within the allowable, except for a short length of about 1,560 ft., where the presence of a rigid bulkhead (needed for two or three years while the pressure tunnel to Chestnut Hill is driven) adversely affected the watertightness of the adjacent joint. Table 3 gives the results of the leakage tests.

It is seen that the tested leakage was generally a fraction of the allowable, showing that the pipe was properly made with well fitting and accurate joints, that it was carefully laid, and especially that the joints were gasketed, caulked and filled in a very satisfactory manner. The manufacturer devoted considerable research, experiment, and effort to furnishing lead gaskets of shape, hardness, and weight that would give the best and tightest joints, and the contractors handled the gaskets carefully and caulked each joint thoroughly in order that leakage might be as low as reasonably practicable.

TABLE 3—TESTED LEAKAGE.

Section	Length Ft.	Internal Diameter, Inches	Leakage G.P.D.	Leakage G.P.D. Per Mile	Leakage G.P.D. Per Inch Per Mile	Remarks
1	9,568	150	1,915	1,057	7.1	Low test head—75 ft.
3	16,524	138	5,264	1,682	12.2	
4	35,820	138	20,788	3,057	22.2	
5						
6	4,904	138	1,484	1,598	11.6	
	8,683	138	7,555	4,606	33.2	Rigid bulkhead causes temporary excessive leakage at one joint.
	1,560	138	8,640	——	210	
	2,268	84	1,831	4,190	49.9	

Both the initial and final caulking were subjected to careful inspection by experienced men to see that the joints were kept clean of silt, sand, etc., before and while each gasket was placed, that each gasket was fitted uniformly to the annular wedge-shaped recess, that the caulking was uniformly, carefully and energetically done, and that any defects in the filling by the gasket were made up by supplementary caulking with lead wool.

The thoroughness with which the mortar fills the caulking recess on the interior of the aqueduct line and the soundness and density of those fillings were evident from inspections made before and after testing. With very few exceptions no cracks were visible on the interior walls of the pipe. Expansion of the line took place by stressing the reinforced-concrete pipe walls rather than by chipping and crushing the joints (most of the mortar filling of joints having been placed during the colder parts of the year).

For purposes of disinfection, the entire aqueduct line was chlorinated by passing through it a long slug of highly chlorinated water, although strict sanitary regulations had been in force, in accordance with the terms of the contracts, both at the manufacturing yard during construction of the pipe units and on the laying jobs during delivery, storage and construction. A solution of sodium hypochlorite, containing about 6,000 lb. of chlorine in 19 m.g. of water, was forced in a slug through the aqueduct at such a rate as to have it in contact with each pipe unit in the line for at least 48 hours. The chlorinated water was wasted slowly at the east end of the line into Norumbega Reservoir and through the blow-off at Charles River in Weston.

Certain observations of a preliminary nature have been made for computing the smoothness and carrying capacity of the completed aqueduct. Although these observations have not yet been sufficiently extensive or complete to draw definite conclusions, they indicate that the interior of the aqueduct is very smooth and has a very high carrying capacity. The value of "*n*" in the Kutter or Manning formulas will not exceed 0.010. Even greater carrying capacities than original studies anticipated are indicated.

The project was built by the Metropolitan District Water Supply Commission, under Eugene C. Hultman, Chairman; Thomas D. Lavelle and Edward J. Kelley, Associate Commissioners; R. Nelson Molt, Secretary; Karl R. Kennison, Chief Engineer (succeeding Frank E. Winsor, deceased); Stanley M. Dore, Assistant Chief Engineer; Charles L. Coburn, Principal Designing Engineer; Walton H. Sears, Mechanical Engineer; and Frederick W. Gow and Coleman C. McCully, Division Engineers. The project was constructed with the aid of a federal P.W.A. grant, and expenditures were supervised by the State Emergency Public Works Commission. The federal Acting Project Engineer was Joseph F. Cairnes, who succeeded Project Engineer Edward E. Murphy (deceased).

Costs

The Portland cement per linear foot of pipe (used in the manu-

TABLE 4—COST OF MANUFACTURE AND DELIVERY OF PIPE PER LINEAR FOOT.

Head, Feet	150-in. Pipe		138-in. Pipe		84-in. Pipe	
	Amount—Ft.	Price	Amount—Ft.	Price	Amount—Ft.	Price
100	9,562	$58.37	35,467	$50.03	87	$24.39
110	——	——	9,492	51.53	—	——
120	——	——	7,839	53.18	48	25.64
130	——	——	4,547	54.53	—	——
140	——	——	1,994	56.23	130	26.89
150	——	——	1,261	57.88	144	27.39
160	——	——	1,438	59.53	144	27.89
170	——	——	831	61.18	35	28.64
180	——	——	2,048	62.73	47	29.14
190	——	——	176	64.43	94	29.64
200	——	——	287	66.93	424	30.39
210	——	——	1,000	68.73	132	30.89
220	——	——	889	70.33	984	31.39
Weighted Average	9,562	$58.37	67,269	$52.63	2,268	$29.90

facture of pipe) averaged 2.65 bbl. for 150-in. pipe; 2.29 bbl. for 138-in. pipe, and 1.03 bbl. for 84-in. pipe. Considering the special connections required, closure pieces, etc., the corrected costs per linear foot for manufacture and delivery of pipe are as shown in Table 4.

The cost of laying per linear foot, exclusive of intake works on Section 1 but including blow-off, air valve, access manhole drainage facilities and other pertinent structures such as fencing is shown in Table 5.

TABLE 5—COST OF LAYING PIPE PER LINEAR FOOT.

	150-in. Pipe	138-in. Pipe	84-in. Pipe
Section 1*	$51.03	——	——
Section 3	——	$29.95	$29±
Section 4	——	30.93	——
Section 5	——	29.02	——
Section 6	——	35.00	30±
Average	$51.03	$31.18	$30±

*Not including work on intake dam and structure.

The unit bid prices for pipe laying are given in Table 6.

TABLE 6—UNIT BID PRICES FOR PIPE LAYING.

Item	Unit	Sections of Aqueduct 1	3	4	5	6
Clearing	acre	$ 75.00	$ 75.00	$100.00	$100.00	$100.00
Grubbing	acre	150.00	100.00	125.00	125.00	150.00
Stripping	cu. yd.	0.70	0.45	0.15	0.15	0.20
Rough grading for pipe delivery	sq. yd.	0.05	0.05	0.11	0.11	0.05
Earth excavation	cu. yd.	0.40	0.30	0.30	0.30	0.30
Rock excavation	cu. yd.	2.50	2.00	2.00	2.00	2.00
Compacted refill and embankments	cu. yd.	0.80	0.40	0.60	0.60	0.50
Refill and embankments	cu. yd.	0.18	0.15	0.16	0.16	0.12
Miscellaneous unconsolidated embankments	cu. yd.	0.15	0.15	0.15	0.15	0.12
Soil Dressing	cu. yd.	0.70	0.60	0.20	0.20	0.30
Seeding and grassing	acre	150.00	100.00	125.00	125.00	200.00
Gravel surfacing for roads	cu. yd.	0.50	0.50	1.75	1.75	1.00
Crushed stone and screened gravel	cu. yd.	3.00	2.50	2.50	2.50	1.50
Paving	cu. yd.	4.00	3.00	3.50	3.50	3.00
Riprap	cu. yd.	2.00	5.00	3.50	3.50	3.00

TABLE 6 (*cont'd*)

Item	Unit	Sections of Aqueduct 1	3	4	5	6
Laying concrete pipe furnished by the Commission	lin. ft.	6.10*	6.00‡	8.00†	8.00†	9.00†
		—	6.00†	—	—	7.00‡
Setting metalwork furnished by the Commission	lb.	0.05	0.02	0.03	0.03	0.01½
Concrete foundations and anchorages for pipe	cu. yd.	7.00	4.25	5.00	5.00	5.00
Reinforced concrete	cu. yd.	14.00	12.00	16.00	16.00	14.70
Miscellaneous concrete	cu. yd.	14.00	12.00	10.00	10.00	15.00
Reinforcing steel	lb.	.04	.04	.04½	.04½	.04
Portland cement	bbl.	2.75	2.40	2.50	2.50	2.48
Steel plate pipe and specials	lb.	.15	.10	.15	.15	.08
Mortar lining for steel pipe	sq. ft.	0.45	0.45	0.25	0.25	0.37
Structural Steel	lb.	0.10	0.10	0.10	0.10	0.09
Galvanizing	lb.	0.04	0.03	0.03	0.03	0.03
Cast-iron pipe and specials	lb.	0.10	0.10	0.07	0.07	0.05
Steel castings	lb.	0.10	0.10	0.20	0.20	0.20
Miscellaneous non-ferrous metals	lb.	0.70	0.60	0.35	0.35	0.40
Stone masonry	cu. yd.	100.00	80.00	65.00	65.00	74.00
Brick masonry	cu. yd.	50.00	50.00	37.00	37.00	27.00
Rubble masonry	cu. yd.	10.00	10.00	5.50	5.50	15.00
Cobble gutters	cu. yd.	2.00	4.00	5.50	5.50	5.00
Corrugated metal culverts	lb.	0.12	0.10	0.08	0.08	0.11
Miscellaneous lumber	Mft.BM	75.00	100.00	90.00	90.00	75.00
Tile pipe 8" or less	lin. ft.	0.50	0.50	0.75	0.75	0.50
Tile pipe 10" to 18"	lin. ft.	2.00	2.00	2.00	2.00	1.50
Replacing highway pavement	sq. yd.	2.50	1.50	2.75	2.75	1.50
Permanent fencing Type A	lin. ft.	0.50	0.40	0.40	0.40	0.32
Permanent fencing Type B	lin. ft.	0.30	0.25	——	——	——
Highway Crossings	each	400.00	400.00	250.00	250.00	200.00
Maintaining flow in open channel	Lump sum	1000.00	——	——	——	——
Intake Structure	Lump sum	2000.00	——	——	——	——
Cleaning up	Lump sum	1000.00	1000.00	1000.00	1000.00	1000.00
Furnishing performance bond	Rate per $1000	15.00	15.00	15.00	15.00	15.00
Sudbury River crossing	——	——	——	6700.00	——	——
Dudley Pond crossing	——	——	——	——	500.00	——
Railroad crossing	——	——	1000.00	——	——	——

*150-in. pipe.
†138-in. pipe.
‡84-in. pipe.

SANITARY SIGNIFICANCE OF THE COLIFORM BACTERIA IN WATER

BY C. A. STUART*

[*Read March 20, 1941.*]

As a member of the American Public Health Association I usually attend the meetings of the laboratory section, particularly when the sanitary standard of water is to be discussed. On several occasions, an engineer has been invited to present to the bacteriologist the engineer's point of view on the subject of safe water supplies. My impression gained from these casual contacts is that engineers, as a group, are most unique or that their representatives on such occasions are very carefully selected. Said representative is always a very affable gentleman radiating personality plus. He always starts his discourse by modestly minimizing the importance of the engineer in water works in general and stresses the importance of the water laboratory in the sanitary control of water. But he inevitably concludes by gently but firmly damning the bacteriologist for often condemning water which the engineer has pronounced safe.

Doubtless engineers occasionally invite a representative of the water laboratory to present the bacteriologist's point of view. While never present at such a meeting, I think I am safe in assuming that the bacteriologist uses the same tactics as the engineer, except of course in reverse order, damning the engineer for even thinking that a water condemned by the laboratory could by any remote chance be safe.

For obvious reasons, engineers would strenuously object if I attempted to call myself an engineer. For similar though perhaps not quite so obvious reasons the water bacteriologists would deny me membership in their clan. If my position in this matter must be defined I can only do so by calling myself an academic bacteriologist, fully realizing that the term academic bacteriologist is enough to make even strong men shudder.

Waterworks men are all aware, sometimes painfully aware of

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the coliform group of bacteria. As waterworks engineers they must have them constantly on their mind, for after all their product is judged primarily by their presence or absence. On the other hand they must at times feel that the water bacteriologist is too coliform conscious since he is frequently stampeded into making dire predictions from a bubble of gas in a tube of lactose broth. Waterworks men are more certain of their position when the gas bubble was generated by a sample from a water that, from a sanitary survey, they have pronounced safe. It is on the thesis that, in some cases at least, they are probably right that I am to report. In order to develop my thesis in what I hope will be a clear manner it is necessary to review briefly our past and present concepts of the coliform group and closely allied bacteria.

In 1885, Escherich isolated from fecal material an organism commonly called *Bacillus coli* of the colon, but whose proper name is now *Escherichia coli* if you are an American or *Bacterium coli* if you are English. Kruse nine years later pointed out that the term *B. coli* was being used to designate a heterogeneous group of organisms widely distributed in the intestinal tract of man and animals. Such organisms may be roughly defined as relatively short, gram negative, non sporing, rods producing acid and gas from lactose and other carbohydrates.

Attempts to divide the *B. coli* group into subgroups began in 1896 and were finally accomplished several years later when it was found that *B. aerogenes* originally isolated by Escherich, differed markedly in its biochemical activities from *B. coli*. Today we recognize two distinct sections or genera of *B. coli;* the *Aerobacter* genus or section of which *aerogenes* is the type species, and the *Escherichia* genus of which *coli* is the type species. These two groups, together with a third group, the "intermediates" which will be discussed later, are collectively known as the coliform group.

The two sections *Aerobacter* and *Escherichia* can be differentiated easily on the basis of their biochemical or so called *IMVIC* reactions. The term *IMVIC* is one of convenience coined by Parr to designate: *I* the indol reaction, *M*, the methyl red, *V* the Voges-Proskauer, and *C* the citrate reaction of the coliform group. The *I* between the *V* and *C* is solely for the purpose of euphony.

Escherichia cultures produce indol, are methyl red positive, are Voges-Proskauer negative in that they fail to produce acetyl methyl

carbinol and citrate negative in that they fail to grow in a medium whose only source of carbon is sodium citrate. They, therefore, possess the *IMVIC* formula of ++−−. *Aerobacter* on the other hand is diametrically opposed to *Escherichia* since the *IMVIC* formula of the former is −−++.

Before the turn of the present century, a few investigators were attempting to establish on a firm basis the important fact that there existed a strong correlation between the normal habitat of the two groups and their biochemical activities. *Escherichia* were found to be typical intestinal parasites of man and animals, whereas *Aerobacter* were infrequently isolated from the intestinal contents but were consistently found in soil and cereals.

Considering the normal habitat of *Aerobacter,* it is not surprising that these organisms occasionally appear in feces for relatively short periods of time, after which they cannot be isolated from the individual even when the feces are inoculated into sodium citrate broth which suppresses the growth of *Escherichia.* Because of the occasional occurrence of *Aerobacter* in feces there was coined the term fecal and non-fecal *Aerobacter,* on the assumption that certain strains were parasitic in man. No such parasitic strains have ever been reported, and consequently all attempts to distinguish between fecal and non-fecal strains of *Aerobacter* have failed.

In 1923, Koser, using a citrate medium, isolated a coliform organism with biochemical reactions intermediate between *Aerobacter* and *Escherichia.* Unlike *Escherichia* it does not produce indol but is methyl-red positive, and unlike *Aerobacter* it does not produce acetyl methyl carbinol but is citrate positive and possesses the *IMVIC* formula −+−+. The normal habitat of these so-called "intermediates" appears to be identical with that of *Aerobacter,* and they are occasionally found in feces but, according to some investigators, less frequently than *Aerobacter.*

Suggestions for classifying the intermediates in the coliform group are varied and in complete disagreement. Some bacteriologists would create a new genus "*Citrobacter*" for such organisms; others would classify them under *Escherichia;* others under *Aerobacter;* and some others would prefer to forget them completely.

Recently the Eijkman test has come back into favor for the differentiation of the sections of the coliform group. This test depends

upon the ability or inability of a coliform strain to produce acid and gas from lactose broth at 45-46 °C. The work of several investigators, working with pure cultures, definitely shows that, with very few exceptions, *Escherichia* cultures produce acid and gas at 45-46 °C more rapidly and more abundantly than at 37 °C, while, with very few exceptions, *Aerobacter* grows sparsely or not at all at this temperature. Moreover, these same investigators have shown that in their Eijkman reactions the intermediates are identical with *Aerobacter*.

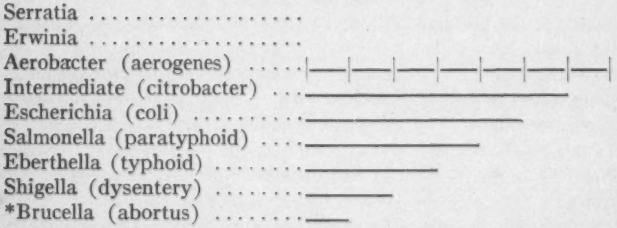
The American Committee on Bacterial Classification, within the year, has drastically altered the classification of the coliform group. Speciation within the group fortunately has been reduced, and, of some 40 ill-defined species in the older classification, only four remain. The Committee recognizes the intermediates as a species *freundeii* under the genus *Escherichia*. While we agree that some distinction should be accorded intermediates we cannot agree that they should be classed as *Escherichia*. Under the present system, one instinctively associates *Escherichia* with organisms whose normal habitat is the intestinal tract, yet the habitat of intermediates appears to be identical with that of *Aerobacter*, and definitely their Eijkman reactions are identical. The fact that *Aerobacter* and intermediates occasionally consort with *Escherichia* in the intestinal tract has tarred the whole coliform group with "the fecal brush". And you will find some text books in bacteriology, used in some of our institutions of higher learning, that do not mention any differences in the normal habitat of the different sections or genera of the coliform bacteria.

It is frequently maintained that, since we cannot determine whether a strain of *Aerobacter* or intermediates isolated from water came to the water from its normal habitat or from a brief visit to the intestinal tract of man, all such organisms must be viewed in the same light as *Escherichia*. This is much like saying that, since some men are thieves, all men must be viewed in the same light. Water pollution is a very serious matter, not only from the public health standpoint, but also from an economic standpoint. It has recently been estimated that the annual capital loss to the United States from water pollution of all kinds is in the neighborhood of seven billion dollars. Under our present economic pressure, we may be certain that any saving made as a result of altering our concepts of the sig-

nificance of the coliform group in water would be greatly appreciated.

So far, I have considered only the work and opinions of other investigators. We, however, are no stranger to the coliform organism. In the past four years at Brown University, we have isolated close to eleven thousand cultures of these organisms from water, milk, soil, feces (human and animal), dust and cereals. All cultures were grouped as *Aerobacter*, intermediates and *Escherichia*. Each group was further subdivided into six types for each group on the basis of additional biochemical reactions. For practical purposes, we do not advocate subdivision of the three coliform sections, but in research work the ability to classify organisms in smaller and more compact groups is often necessary for a complete picture of the group as a whole. In addition to the coliform group, several hundred cultures of paracolon and true intestinal pathogens have been studied. From these investigations, we have returned to an old concept of a sort of evolutionary trend in the normal and pathogenic bacteria encountered in the intestinal tract of man. The accompanying figure, showing a so-called metabolic gradient in the intestinal bacteria, will convey what we mean more clearly.

METABOLIC "GRADIENT"
WITHIN
THE FAMILY ENTEROBACTERIACEAE



Divisions in the line opposite *Aerobacter* may be taken to represent in order, from left to right, the ability to ferment mono-saccharides, di-saccharides, tri-saccharides, to produce acetyl methyl carbinol, to liquefy gelatin, etc. In other genera, the shorter the line

*Now in family Parvobacteriaceae.

opposite the genus, the lower the metabolic activities of that genus. When specific substances such as particular carbohydrates are used in the base line, the gradient breaks down in a few places, but the general concept that we are dealing with different degrees of specialization among the several genera still holds true.

Aerobacter are capable of producing acid and gas from practically all carbohydrates, polyhydric alcohols and glucosides, of producing acetyl methyl carbinol and liquefying gelatin. *Escherichia* produce acid and gas from many carbohydrates, few alcohols and glucosides, do not produce acetyl methyl carbinol, nor liquefy gelatin. *Salmonella* (paratyphoid organisms) produce acid and gas, for the most part only from the less complex carbohydrates whereas *Eberthella* (typhoid) and *Shigella* (dysentery) produce acid only from similar sugars. *Brucella* (undulant fever organisms) utilize the monosaccharide dextrose to some extent but are incapable of producing a visible acid reaction.

A similar gradient is shown in the growth of these organisms on a medium containing for example asparagin as the only source of nitrogen. *Aerobacter* grow abundantly, *Escherichia* produce a fair growth, while the remainder grow sparsely or not at all. In a medium with sodium citrate as the only source of carbon, only *Aerobacter* and intermediates grow.

We have studied the antigenic relationships of the coliform bacteria and find a similar gradient which we will not discuss in detail. In other words we are dealing with a group of bacteria which are all interrelated but which show varying degrees of specialization. *Salmonella, Eberthella, Shigella* and *Brucella* are definitely parasitic and pathogenic; *Escherichia* parasitic but not pathogenic in its normal habitat; while *Aerobacter* is not parasitic and its normal habitat is not man, though it occasionally occurs as a "bird of passage" in the intestines.

Assuming an evolutionary trend in this group, one naturally wants to know more about the possible progenitor of *Aerobacter*, and here we encounter an interesting and instructive possibility. The genus *Erwinia* is a well known but poorly delineated group of organisms. They are natural plant pathogens causing blights and wilts. They are found in soil, and after killing their host they must return to the soil

in great number and consequently must find their way into water at times. Morphologically, they are indistinguishable from the coliforms. They ferment many substances including lactose with the production of acid and gas. For the most part their *IMVIC* reactions are those of *Aerobacter* and intermediates though some have the ++ – – reaction of *Escherichia*. The only way in which an *Erwinia* culture can be satisfactorily classified is by its pathogenicity for plants. Acid and gas in a lactose broth tube may be indicative of *Erwinia* as well as coliforms.

Going farther back we find *Serratia*. Ordinarily, these organisms produce a red pigment and produce only acid in many carbohydrates. Non-pigmented strains of these organisms are not infrequently encountered, and occasionally a gas-producing strain is found. A non-pigmented gas-producing strain cannot be distinguished by any known method from *Aerobacter* since the *IMVIC* reactions of all the *Serratia* strains studied by us are – – ++. We have isolated non-pigmented gas-producing *Serratia* from water, milk and human feces.

It is evident, then, that the appearance of gas in a lactose broth tube in 48 hours is not an absolute criterion of the presence of a member of the coliform group and even less a criterion of fecal pollution. Experience sometimes confirms this statement. In the summer of 1939 we surveyed a number of polluted wells. Four appeared to be safe. Frequent sampling throughout the summer showed coliforms ranging from one in one milliliter to fifty per milliliter. *Aerobacter* and intermediates were the offenders in these cases, with *Aerobacter* predominating. Samples of soil and grasses in the vicinity of the well showed not only the same genera of coliforms as were isolated from the water but the same types within the genus.

Three municipal water supplies in a neighboring state considered safe by the engineer but not infrequently failing to meet the U. S. Treasury Standard were extensively examined over a period of one year. The results were disappointing since *Escherichia* were found in all three supplies, together with a predominating number of *Aerobacter* and intermediates. Yet for over a period of twenty years no epidemic has ever been traced to these waters, and only one epidemic of gastroenteritis was even considered as possibly water borne. If this record is maintained for another twenty years it is possible that

even the Public Health Service Standard will consider these waters safe.

The disappointing results obtained from the municipal water supplies just mentioned should not deter waterworks men from insisting on further information concerning the organisms involved when the laboratory condemns, as polluted, a supply which after a thorough and careful survey has been pronounced safe by a competent sanitary engineer.

THE BACTERIOLOGICAL DETECTION OF HEALTH HAZARD IN WATER SUPPLIES†

BY W. L. MALLMANN*

[*Read April 3, 1941.*]

It has been definitely proved that untreated sewage-laden water carries organisms capable of producing gastro-enteric diseases. A review of the numerous water-borne epidemics recorded in the literature demonstrates this fact conclusively. No one would question the statement that untreated sewage-polluted water is potentially a public health hazard.

Adequately treated sewage-polluted water can be rendered free from health hazard, irrespective of the amount of sewage pollution, as witnessed by safe supplies in Michigan obtained from heavily polluted sources. The use of such raw water is not good sanitary practice; however it is undoubtedly true that most any water by proper processing can be rendered safe.

Unfortunately there are many sewage polluted waters that are inadequately processed and, although they may meet our sanitary standards for quality at most times, they are distinctly marginal waters. Such waters are not processed sufficiently to carry a safety margin to care for unusual pollutional loads. For example, lake or river waters that are chlorinated but not filtered may receive unusually heavy and sudden sewage contamination. The chlorine feed-rate may be insufficient, and disease bacteria may enter the distribution system. Such waters are potential health hazards.

The effectiveness of treatment in the elimination of health hazards is generally based on the fact that the use of treated marginal waters has not resulted in the appearance of gastro-enteric diseases in the users of the water and the fact that the sanitary quality of the water has been demonstrated to be safe according to our established bacteriological standards. During the past several years, there have occurred in this country many epidemics of diarrhea or mild dysentery that have been charged by epidemiological evidence to the consumption of treated sewage-polluted water. It is not the purpose of this paper to discuss

†A paper read before a joint meeting of the New England Health Institute, the New England Water Works Association, and the New England Sewage Works Association.

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the validity of the data purporting to show that the water was the cause of the epidemics reported. This paper is predicated, however, on the thesis that, at least in part, some of these epidemics were caused by inadequate treatment. An examination of the bacteriological reports of some of these waters, collected during the period of the epidemic, fails to show the presence of coliform organisms. If these reports are accepted as evidence that the waters were free from health hazard, then it follows that the treatment was adequate and, further, that the operators had used all due precautions in the operation of the treatment plant.

It would appear that our sanitary yardstick, the bacteriological test for coliform organisms, lacked sensitivity in that it had failed to indicate the presence of a health hazard. There is evidence in the literature that such may be the case. There are a number of reports in the literature showing that when lactose-broth tubes were planted with marginal water, *e.g.* treated sewage-polluted water, gas production occured if the incubation period was prolonged over the customary forty-eight hour period. The organisms isolated from these tubes in subsequent transplantings into lactose broth were classified as slow lactose fermenters, but they were unquestionably members of the coliform group. Considerable discussion has appeared in the literature on the significance of these organisms. Again it is not the purpose of this paper to discuss the epidemiological significance of these organisms.

We have been interested for some time in determining, at least in part, some of the factors causing these bacteria to attack lactose slowly. It has been observed by many workers that many of these organisms acquire or recover the property of rapid gas production from lactose after they have been kept in culture-media for some time. This has been the experience of the writer. These observations would seem to indicate that environmental factors had caused a change in the bacteria and that these organisms were, in a sense, typical coliform bacteria. The survival of these bacteria is just as much an indication of inadequate treatment as is the presence of typical coliform organisms. Since treated marginal waters appear to be responsible for outbreaks of diarrhea and mild dysentery and since these slow lactose fermenters are occasionally found in these waters, it would seem reasonable to believe that they should be absent in an acceptable supply.

The writer has questioned seriously the value of standard lactose broth as an enrichment medium for culturing coliform organisms, particularly if they had been attenuated by chlorination or if they appeared in the water in minimal numbers. The fact that coliform organisms appeared in the tubes after 48 hours of incubation, as evidenced by gas production, seemed to justify this belief. When bacterial cells are introduced into a new medium, such as occurs when portions of water containing bacteria are introduced into standard lactose broth, the bacterial cells are more or less dormant for a period of time during which they are adjusting themselves to their new environment. If they are successful in making the adjustment, then reproduction begins. As soon as reproduction is completed, a new critical period in the life of the cells occurs. A number of papers have appeared which show that these young cells, at this time, are very susceptible to adverse conditions. Many cells die. If we were to substitute a more favorable medium than standard lactose broth, it is conceivable that many of these cells, that ordinarily would die, would continue to grow and reproduce. Furthermore, if the medium were more favorable to growth, the period of adjustment previous to reproduction would be shortened. With these points in mind, the various nutrients entering into the composition of nutrient broth were tested using minimal dosages of attenuated coliform organisms. The final medium selected as representing the most favorable conditions was a tryptose lactose broth. Table 1, taken

TABLE 1[1]—COMPARISON OF STANDARD LACTOSE BROTH AND LACTOSE TRYPTOSE BUFFER BROTH FOR RATE OF GROWTH OF *Escherichia Coli*, No. 161.

Medium	pH	Initial	3 hours	6 hours	9 hours	12 hours
				Series No. 1		
				Number of Bacteria per ml.		
Standard*	6.55	51	960	79,000	7,750,000	229,500,000
Tryptose†	6.8	64	5,885	1,440,000	970,000,000	2,205,000,000
				Series No. 2		
Standard	6.8	29	49	2,300	62,000	360,000,000
Tryptose	6.8	49	420	80,300	115,000,000	2,070,000,000
				Series No. 3		
Standard	6.8	25	41	455	25,000	1,235,000
Tryptose	6.8	27	72	1,785	170,000	13,600,000

[1]Table 12—Darby C. W., and Mallmann, W. L. Studies on Media for Coliform Organisms. Jour. Am. Water Works Assoc., 31, 689 (1939).

*Standard: Bacto-lactose broth, A.P.H.A.

†Tryptose, medium consists of: Bacto-tryptose 2%, lactose 0.5%, K_2HPO_4 0.4%, KH_2PO_4 0.15%, and NaCl 0.5%.

from a publication by Darby and Mallmann (1), shows that not only do the bacteria grow more rapidly in this medium, but that they reach the peak of their growth phase in a much shorter period of time. Gas production would of course appear earlier in this medium as shown in Table 2.

TABLE 2—THE INFLUENCE OF THE MEDIUM ON THE RATE OF GAS PRODUCTION OF *Escherichia coli*, No. 161.[2]

Medium No.	Tube No.	Time of appearance of gas				
		9.5 hours	10.5 hours	11.5 hours	12.5 hours	13.5 hours
1	1	+	10	40	60	90
	2	+	15	40	60	80
	3	+	10	40	60	80
	4	+	15	40	70	80
	5	+	20	50	70	90
2	1	—	—	—	5	20
	2	—	—	+	5	10
	3	—	—	—	10	15
	4	—	—	—	5	10
	5	—	—	+	10	20

Medium 1: Bacto-tryptose 2%, lactose 0.5%, NaCl 0.5%, K_2HPO_4 0.4%, and KH_2PO_4 0.15%.
Medium 2: standard Bacto-lactose peptone broth.
Inoculum: 580 organism per tube.

[2]Data taken from Table 13—Darby, C. W. and Mallmann, W. L. Studies on Media for Coliform Organisms. Jour. Am. Water Works Assoc., 31, 689 (1939).

In 1932, Cowls (2) reported that the addition of sodium lauryl sulfate to standard lactose broth produced a medium selective for the coliform group. Later, in a study made by Mallmann and Darby (3) on the toxicity of such bacteriostatic agents as fuchsin and brilliant green, it was found that sodium lauryl sulfate did not exhibit any inhibitory action on the coliform group. This was not true of other bacteriostatic agents studied. Because of its lack of inhibitory effect on the coliform group, this material was incorporated into the tryptose broth thus making the medium a selective enrichment medium. The medium was supplied to a number of water purification plants for test, using standard lactose broth in parallel. Some very interesting results were obtained. When the medium was tested on waters polluted with raw-sewage, the number of positive tubes in the two media was ap-

proximately the same. This result would be expected because of the presence of excessive numbers of coliform organisms in the water and the lack of attenuation of the organisms. When the medium was used on marginal waters, in which coliform organisms were seldom isolated but in which spore-forming lactose splitters were common, it failed to show gas, whereas the standard lactose broth showed gas production frequently but seldom confirmed. In a series of tests in six filtration plants, standard lactose broth was positive in 238 tubes with only three confirming and the lauryl sulfate lactose tryptose broth was positive in three tubes and confirmed in all of them.

During the course of studies on this series, it was frequently found that the colonies on eosin-methylene-blue agar, obtained from positive standard lactose broth tubes, were negative when transplanted again into lactose broth, but that they were positive when they were transplanted into tryptose lactose broth. These organisms were identified as coliform organisms. Table 3 shows the comparative confirmation tests by parallel plantings from eosin-methylene-blue plates to standard lactose broth and tryptose lactose broth. In Table 4 are presented the colon indices on those samples. It will be observed that the average colon index was 1.3 for lactose broth confirmation whereas with tryptose lactose broth the average was 5.0. These data indicate that the gas produced in the primary lactose tubes was due to the presence of coliform organisms.

When the coliform organisms just reported were smeared on the surface of an eosin-methylene-blue agar plate, the organisms in the resulting colonies, which appeared atypical, were non-gas-producing when they were planted into lactose broth for refermentation. If the organisms from the same colony were transplanted to tryptose lactose broth, gas production was obtained in 48 hours. It is thus readily seen that the eosin-methylene-blue agar had caused a suppression of the enzymes causing lactose fermentation. This is not unexpected, since the writer has observed that the introduction of typical coliform organisms isolated directly from the intestinal tract, into brilliant-green broth or agar frequently suppressed their ability to ferment lactose. Several years ago, the writer isolated approximately 60 strains of organisms that were thought to belong to the paratyphoid group and later found that they were coliform organisms when they recovered their ability to ferment lactose with gas production.

TABLE 3—COMPARATIVE CONFIRMATION TESTS BY PARALLEL PLANTINGS FROM E.M.B. PLATES TO STANDARD LACTOSE BROTH AND TRYPTOSE BROTH.[3]

Sample No.	Gas in 10 cc. Portion Lact. Broth					E.M.B. from Lact. Broth					Lact. Broth from E.M.B.					Trypt. Broth from E.M.B.					Trypt. Broth from Lact. Broth				
	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
43	3	3	3	3	—	+*	+	+	+		—	—	—	—		10	20	20	10	—	+	+	+	+	
44	5	3	3	3	—	+	+	+	+		20	—	—	—		30	30	20	15			+	+	+	
46	20	5	1	1	1	E	+	+	+	—		2	—	—			30	5	—					+	
65	1	5	5	3	5	+	+	+	+	+	—	2	2	—	—	10	10	—	5	5	+	+		+	+
89	3	3	3	3	3	+	+	+	+	+	—	—	—	—	—	10	20	10	5	5	+	+	+	+	+
95	60	90	40	70	90	A[2]	Cl.[3]	+	+	E			—	—				2	40				+	+	
96	40	15	30	10	40	+	Cl.	Cl.	Cl.	Cl.	—					—					—				
104	20	50	40	—	—	+	+	+			—	—	—			10	4	3			+	+	+		
140	—	3	3	3	5		+	+	+	+		—	—	—	—		3	5	3	5		+	+	+	+
187	2	—	10	—	2	+		A			—				—	3				3	+				+
261	50	50	50	50	—	E	+	+	+			—	—	—			3	3	2			+	+	+	

[3]Table 11—Mallmann, W. L. and Darby, C. W. Uses of a Lauryl Sulfate Tryptose Broth for the Detection of Coliform Organisms. Am. Jour. Pub. Health, 31, 127 (1941).

*+ indicates atypical colonies.

[2]E indicates typical *Escherichia* colonies.

[2]A indicates typical *Aerobacter* colonies.

[3]Cl. indicates presence of *Clostridia*.

TABLE 4—COMPARATIVE COLON INDICES OBTAINED BY STANDARD METHODS OF TRANSPLANTING EOSIN-METHYLENE-BLUE COLONIES TO TRYPTOSE LACTOSE BROTH.[4]

Sample No.	Gas Index	Confirmed Index	Tryptose Broth Confirmation
43	8	0	8
44	8	2	8
46	10	2	6
49	4	0	0
65	10	4	8
75	2	0	2
89	10	0	10
95	10	4	4
96	10	0	0
103	2	0	2
104	6	0	6
140	8	0	8
187	6	2	4
200	10	0	4
230	2	2	2
234	6	2	6
261	8	2	8
Average	7.05	1.3	5.0

[4]Data adapted from Table 13—Mallmann, W. L. and Darby, C. W. Uses of a Lauryl Sulfate Tryptose Broth for the Detection of Coliform Organisms. Am. Jour. Pub. Health, 31, 127 (1941).

These data have been presented, not as a means of introducing the reader to a new medium or a new procedure for the demonstration of coliform organisms in marginal waters, but primarily to show that the presence or absence of coliform organism in marginal waters is arbitrarily based on the selection of the medium used for their isolation and not on their actual presence or absence. Such data would seem to indicate a greater need of close supervision of marginal water supplies to be sure that epidemics of water-borne diarrhea and dysentery are avoided.

The protection of a water supply and the supervision of the sanitary quality of the water are of the utmost importance to the consumer at all times. The plant operator should have a full realization of the job intrusted to him and should exercise every means at his disposal to be sure that the output of his plant is safe twenty-four hours a day and three hundred sixty-five days a year. The introduction of our

defense program should not make any difference in the operation of the plant, except that perhaps greater care and vigilance should be observed where the source of water may be affected or the plant overtaxed in output. The plant operator should watch the source of water more closely, particularly where new water, industrial and domestic, may be introduced as a result of industrial and defense developments on water sheds. Plant expansions should be made in anticipation of increased water demands, so that plant operations may not be overburdened and ineffective treatment result.

The defense program is not introducing any new problem in the handling of water supplies and sewage disposal, but it is creating sanitary problems and health hazards by the speed of industrial developments that are not being supplemented by equal speed in the development of proper and adequate water supplies and sewage disposal systems. Defense plants, located in distinctly rural areas, are mushrooming into great employment centers, and workers and their families are crowding into these areas without adequate housing and sanitary conveniences. Unless drastic steps are taken to protect the public health, serious epidemics may result.

Conclusion

Water works operators, particularly those handling marginal water supplies, should exercise extreme care in making bacteriological tests for coliform organism and should attempt to make their treatment processes so effective that no coliform organism are ever found in the finished water. By so doing, water-borne disease will be further minimized.

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SOME CHEMICAL ASPECTS OF BREAKPOINT CHLORINATION

BY A. E. GRIFFIN* AND N. S. CHAMBERLIN**

[*Read December 19, 1940.*]

When chlorine is added to water until the first residual appears, the residuals produced after further additions of chlorine will follow one of the curves, or some modification of one of the curves shown in Fig. 1,

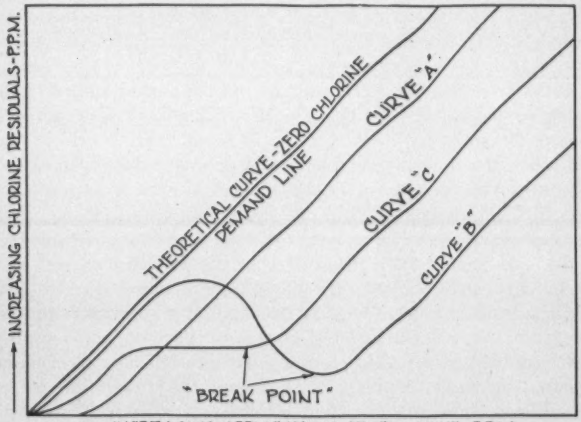

FIG. 1-TYPICAL CHLORINE RESIDUAL CURVES OBTAINED WHEN WATER IS TREATED WITH INCREASING AMOUNTS OF CHLORINE.

FIGURE 1.

rather than the straight line proportional to the applied chlorine, as might be expected (Fig. 1).

In general, residuals in clear colorless water, free from organic matter, all forms of ammonia, algae and other chlorine-absorbing ma-

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terials, will yield curves of Type A, Fig. 1. In such water, the breakpoint is seldom clear, and its recognition may be extremely difficult. To secure maximum results under these conditions, particularly where tastes and odors must be taken into consideration, the breakpoint can be sharpened by the introduction of a small amount of ammonia ahead of the chlorine.

Of greatest interest is a water in which chlorine reacts as indicated by the shape of Curve B, Fig. 1. This type of curve occurs where free ammonia alone, or free ammonia with nitrogen in other forms, exists in the water. Such a condition is most frequently found in highly polluted waters (waters polluted by sewage, industrial wastes, creamery wastes, etc.), and the magnitude of the hump and dip in the residual curve will usually increase as the extent of pollution rises. It should be noted particularly that, in waters of this type, the original taste and odor are greatly intensified in the early stages of chlorination but are reduced with decreasing chlorine residual as the breakpoint is approached and disappear altogether in most cases at the breakpoint.

Waters high in albuminoid ammonia, or suspended organic matter, will usually yield curves similar to Curve C, Fig. 1. Often, such a breakpoint will amount to little more than an off-set in the residual curve whereas, at other times, it assumes the character of a plateau. In the field, this is often referred to as the "plateau" or "off-set" curve. In either case, under the influence of chlorine, tastes and odors will be at a minimum at or just beyond the breakpoint. From this point on, as chlorination is continued, chlorine odors will become increasingly apparent, but these may readily be removed through the use of suitable dechlorinating agents. Further, at or beyond the breakpoint, bacterial reduction may often be greater than otherwise possible, and at the same time waters of acceptable quality, as regards taste and odor characteristics, will be produced.

The knowledge that residual curves and results, as indicated, will be produced following chlorination is not new, for numerous investigators have repeatedly reported such characteristics. The literature, however, fails to reveal any comprehensive data on the nature of the reactions involved. Calvert (1), during the winter of 1940, established the fact that the breakpoint was a direct function of the amount of ammonia present and hinted at the possibility that, where such a phe-

nomenon takes place, there might be a lowering of the ammonia content of the solution. Further, it has long been known that, under alkaline conditions, the halogens will oxidize certain forms of nitrogen.

With these preliminary data as a background, a series of experiments was undertaken to study the effect of chlorine on water containing a given amount (0.50 p.p.m.) of ammonia-nitrogen (in the absence of interfering substances) at pH 5.0, 6.0, 7.0, 8.0 and 9.0. Throughout the work, ammonia-free distilled water was used, suf-

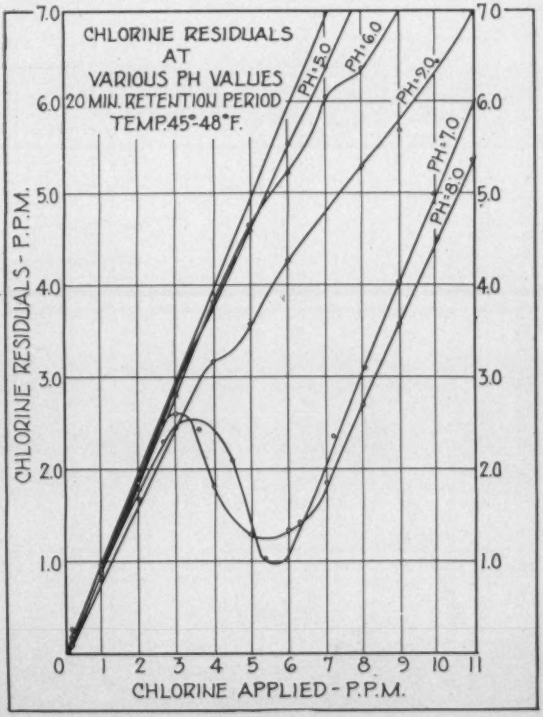

FIGURE 2.

ficiently buffered to prevent any appreciable drop in pH regardless of the amount of chlorine added. All pH determinations were made with the glass electrode. Temperatures in all instances were held between 45 and 48°F. The chlorine remaining at the end of 20-min., 2-hr., and 24-hr. intervals, was determined by titration which starch iodide without the addition of acid. Residual ammonia at the end of 2-hr., and 24-hr. intervals was determined by distillation and Nesslerization according to Standard Methods (2). Prior to distillation, the chlorine was neutralized by the addition of sulphurous acid.

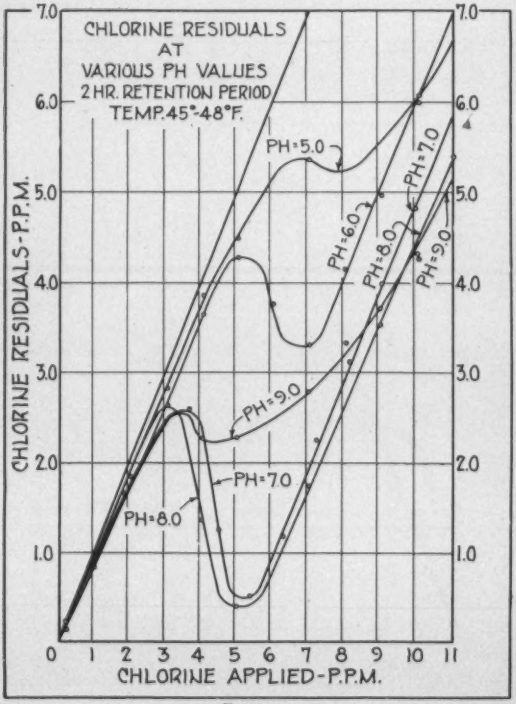

FIGURE 3.

Under these conditions, there was little loss of chlorine after a 20-min. period of contact at pH 5.0 (Fig. 2), with the result that this chlorine residual curve showed little semblance to a breakpoint curve. At the end of 2 hr. (Fig. 3), enough loss in chlorine was recorded to produce the outline of a breakpoint, and by the time 24 hr. (Fig. 4) had elapsed, the reactions had progressed far enough to produce a typical breakpoint curve. Although less chlorine was lost over the 24-hr. period

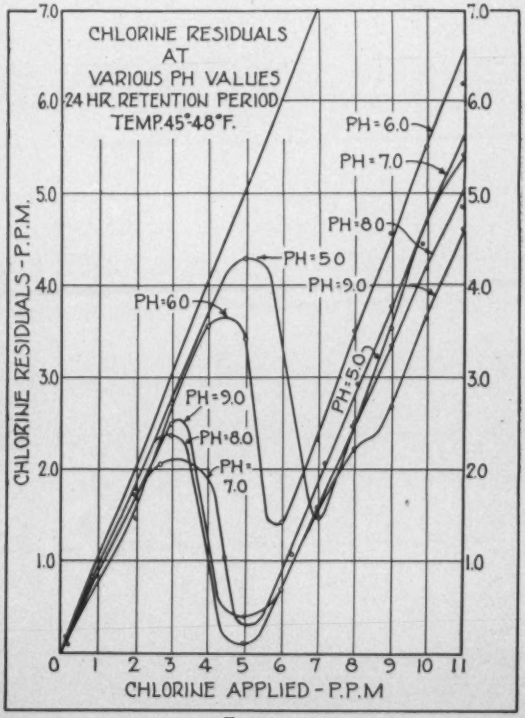

FIGURE 4.

at pH 5.0 and 6.0 than at pH values of 7.0, 8.0 and 9.0, it seems reasonable to assume that, had the experiment been extended another 24 hr., the residual curves at all pH values would have closely approximated each other. On the other hand, the formation of the breakpoint at pH 7.0 and 8.0 had progressed rapidly within the first 20 min. and had apparently gone to completion by the end of 2 hr. These results (Figs. 2, 3 and 4) suggest that the formation of the breakpoint curve requires at least 20 min. at pH 7.0 and 8.0, and up to 24 hr. for the other pH

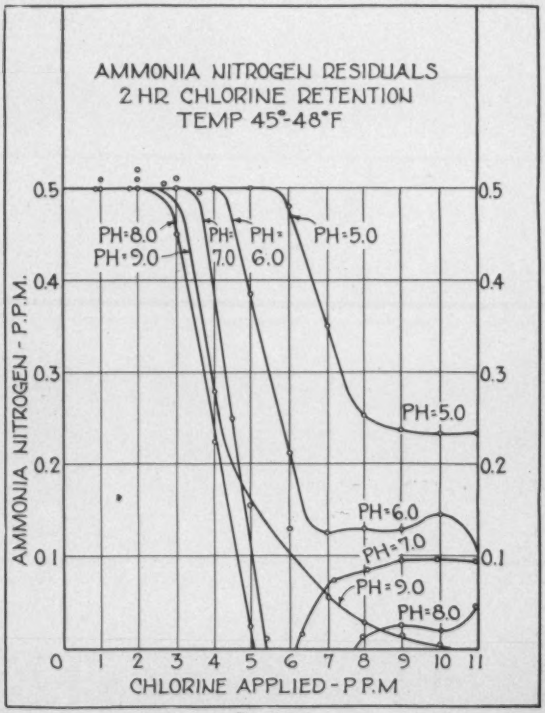

FIGURE 5.

values studied. They further suggest that the optimum pH for the reaction between chlorine and ammonia lies in the neighborhood of 7.0 and 8.0. Plant practice has indicated, however, that this optimum range may be extended to include a pH range from 6.5 to 8.5 (Figs. 2, 3 and 4).

That the removal of ammonia is also progressive with the increase in pH and time is shown in Figs. 5 and 6. Maximum removal of ammonia in all cases occurred at the point of minimum residual beyond

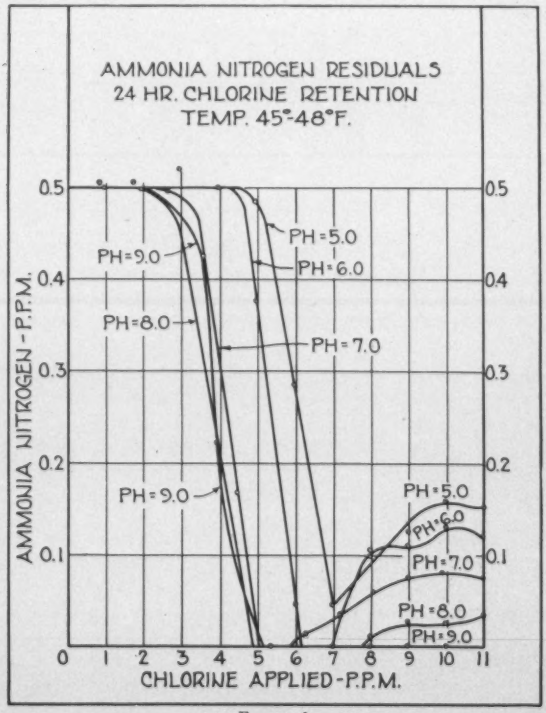

FIGURE 6.

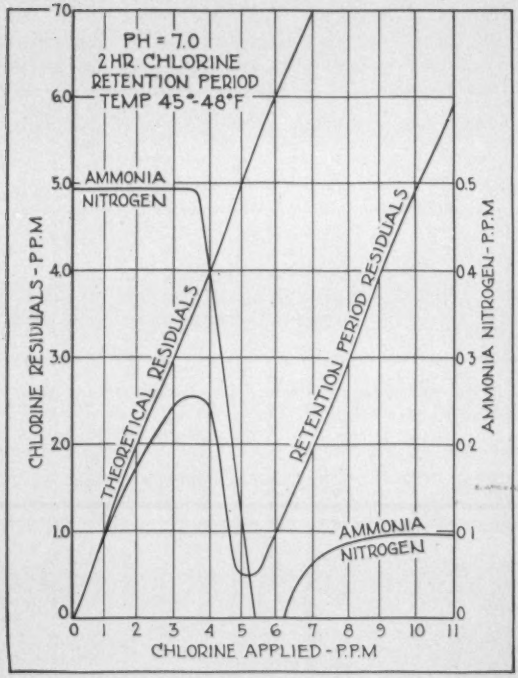

FIGURE 7.

the hump, as exemplified in Fig. 7. Here there was no loss in nitrogen until the chlorine residual had reached its maximum. From this point on the nitrogen, as well as the chlorine, decreased rapidly, with increasing additions of chlorine, until slightly more than 5.0 p.p.m. had been added. At that point, the chlorine residual had fallen to a minimum, and the nitrogen had entirely disappeared. With further additions of chlorine, the residual curve rose in direct proportion to the dosage curve, and slight amounts of nitrogen reappeared. Similar comparisons of

residual chlorine and ammonia can be obtained by superimposing Figs. 2 and 3 on Figs. 5 and 6.

Chapin (3, 4) and others, when working with relatively high concentrations of chloramines (1,000-2,000 p.p.m.) demonstrated that the ratio of nitrogen to chlorine required to completely remove the ammonia was 1:7.5 by weight. When, however, more dilute solutions were used (about 70 p.p.m.) this ratio changed to 1:8.5 by weight. In our experiments this ratio was found to be very close to 1:10 by weight.

pH	p.p.m. N consumed	p.p.m. Cl consumed	N:Cl₂ Ratio by Wt.	N:Cl₂ Ratio by Mol.
6.0	0.361	3.64	1:10.08	1:2.01
7.0	0.495	5.00	1:10.12	1:2.02
7.5	0.500	5.04	1:10.08	1:2.02
8.0	0.490	4.67	1:9.54	1:1.91
Av.	0.461	4.59	1:9.96	1:1.99

This ratio of 1:10, however, is good only for distilled water under the conditions that have been described. In practice, due to variations in amounts and types of impurities, temperature and contact, this ratio usually deviates considerably from this figure and usually toward a greater ratio. The curves in Fig. 8, obtained by the addition of varying amounts of chlorine to four different waters, each containing the same amount of ammonia-nitrogen (0.5 p.p.m.) are given as examples. Curve A, obtained by treating distilled water with chlorine, reacts according to theory, whereas Curves B, C, and D, representing three natural waters obtained from different sources, deviate considerably from theory. In practice, this ratio tends to spread out toward 1:15 or beyond. A few cases are on record where the ratio was as great as 1:25 (Fig. 8).

In the course of this work there were indications that the residuals within the hump part of the curve were due to chloramines, and that the residuals beyond the break were largely due to chlorine alone. When ortho-tolidin develops a color very slowly, taking some time to reach maximum, this behavior has generally been accepted to be due to the presence of chloramines. It is a known fact that ortho-tolidin develops a color in a flash, reaching a maximum in a very short time, with chlor-

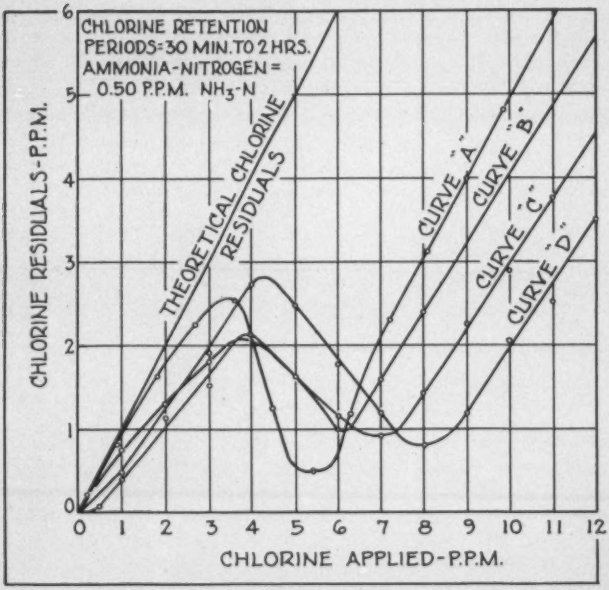

FIGURE 8.

ine alone. In these particular tests, the development of color in the hump part of the curve gave no flash with ortho-tolidin and was slow in appearing, whereas, beyond the break, some color was developed instantaneously with ortho-tolidin and reached a maximum in a very short time.

This has further been substantiated by bacteriological studies, such as those by Levine (5). Figure 6 shows that there is little or no loss in nitrogen up to the maximum production of a residual and that up to this point NH_3-N is in excess of that theoretically required to completely react with the chlorine present to form chloramines at the pH in question. As soon, however, as this theoretical ratio is approximately at the top of the hump (Fig. 7), other reactions begin to take

place, resulting in a loss of ammonia and chlorine. That this could be expected has been shown by Raschig (6), Rideal (7), and Race (8), who have indicated that the reaction between chlorine and ammonia, wherein chloramines are formed, is well defined in the presence of an excess of ammonia, but that where the chlorine is in excess such formation may be influenced by many factors.

This leads to the conclusion that so long as ammonia is in excess, which is the condition prevailing up to the maximum residual in the hump part of the curve, the chloramine reactions will progress according to Chapin's formulae and that it is only where chlorine is in excess of that theoretically necessary for the formation of chloramines that breakpoint reactions take place.

Although Chapin has indicated that in a 0.1-molar solution of ammonia, particularly where ammonia is in excess, mono-chloramines alone will be found at pH 8.5 or above, that between pH 4.5 and 5.0 only di-chloramines will be found, and that between pH 5.0 and 8.5 mixtures of the two will prevail, little information is available regarding the behavior of chlorine and ammonia in the concentrations (0.5 p.p.m. or 0.000036-molar) ordinarily encountered in practice.

According to equations (1) and (2), twice as much chlorine is required for the formation of di-chloramines as for mono-chloramines. Thus, in a solution containing 0.5 p.p.m. of NH_3-N, 5.0 p.p.m. of chlorine are required in equation (1) to form di-chloramines against 2.5 p.p.m. in equation (2) to form mono-chloramines.

$$NH_3 + 2\ Cl_2 \rightarrow NHCl_2 + 2\ HCl \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

0.5 part (N) 5.0 parts di-chloramines

$$NH_3 + Cl_2 \rightarrow NH_2Cl + HCl \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

0.5 part (N) 2.5 parts mono-chloramines

In Fig. 9, the nitrogen and chlorine residuals have been plotted for pH values of 5.0 and 9.0 for the 24-hr. chlorine retention periods. Under these conditions, at pH 5.0, there is little loss of nitrogen, and the residual is at its approximate maximum at that point where 5.0 p.p.m. of chlorine have been added. At pH 9.0, however, the peak in residual is reached upon the addition of only half that amount of chlorine, namely 2.5 p.p.m. When more chlorine is added at pH 9.0, both the nitrogen and chlorine rapidly recede. Having previously proved that

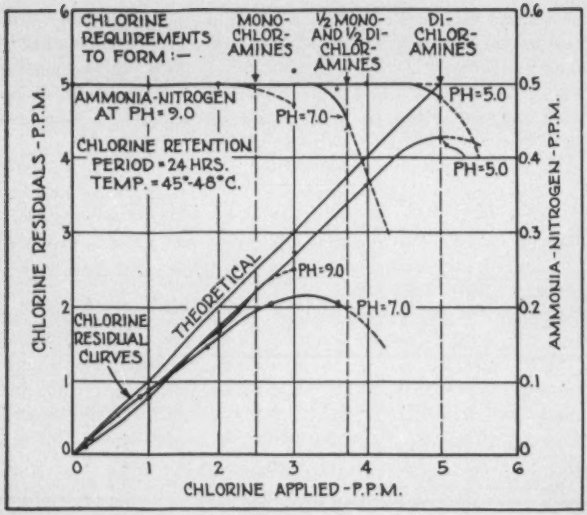

FIGURE 9.

the residuals in the hump part of the curve are chloramine in nature, the inference from the data plotted in Fig. 9 is: (1) that when 2.5 p.p.m. of chlorine are added to a water containing 0.5 p.p.m. of NH_3-N buffered at pH 9.0, mono-chloramines are formed and that further additions of chlorine only tend to destroy both the N and chlorine, and (2) that at pH 5.0 the chlorine additions must be carried to twice this amount, namely, 5.0 p.p.m. before any loss of N or chlorine becomes evident, thus indicating the formation of di-chloramine at pH 5.0. These figures then, check with the formulae and tend to confirm prior assertions that mono- and di-chloramines are formed by ammonia and chlorine at the concentrations ordinarily encountered in practice.

Summarizing, we find that: (1) the residual curves obtained by the chlorination of water will follow one of three general types depending upon the character of the water; (2) the magnitude of the breakpoint curve and the amount of chlorine required to produce it

depend to a large extent upon the amount of free ammonia (NH_3-N) in the water; (3) at the breakpoint both N and chlorine, as residual chlorine, are lost; (4) the speed and extent of the reaction are dependent to a great extent on the pH of the solution; (5) the optimum pH for plant practice lies between 6.5 and 8.5; (6) the time of contact is of considerable importance in the completion of the reaction; (7) the ratio of N to chlorine at the optimum pH to reach the breakpoint is 1:10, which may extend to 1:25, or beyond in natural waters; (8) chloramines are formed in the area of the "hump" and are more stable in the presence of an excess of ammonia than in an excess of chlorine; and (9) apparently mono-chloramines are formed at pH 9.0 and di-chloramines at pH 5.0.

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WATER

BY ROBERTA WHITEHEAD*

[*Received March 18, 1941*]

My name is Water; I have sped
Through strange, dark ways, untried before
By pure desire of friendship led,
Cochituate's ambassador.
He sends four royal gifts by me
Long life, health, peace and purity.
—*James Russell Lowell*, 1848.

The following children's story, "Water," represents reading material used in the second grade of Shady Hill School, Cambridge, Massachusetts, as part of their study of the way in which Cambridge is supplied with utilities. The story summarizes a variety of classroom discussions and experiments, and a visit to the local reservoir and purification plants. These activities included the construction of a model-table watershed and reservoir, a standpipe and distribution system, and simple sand filters. The action of ferric alum floc upon silted and colored water was also studied.

The general principles of precipitation, runoff, collection, purification and distribution are easily understood by seven-year-old children. Further, the problems involved in city water supply have high intrinsic interest. The success of this project suggests the degree to which very young children may understand the importance of adequate, safe water in their community. With the exception of a few technical terms, the words fall within the normal reading range of seven-year-old children.

*Teacher, Shady Hill School, Cambridge, Mass.

WATER

When you drive along in the country
not very far from a big city
you often see large ponds or lakes.

Some of these are lakes
where people swim
and sail and fish.
But some of them are reservoirs.

A reservoir looks just like a pond or lake
but there is something special
about it.

No one ever swims in a reservoir
or fishes in it
or rows a boat on it.

No one builds a house very close to it.

Because the water in a reservoir
is water that people will drink
and so it must be kept clean.

Whenever you see a reservoir
you will see a sign
that says something like this:

No Fishing, Swimming or Picnicking
Allowed Here.

And you know why that is.

The water in the reservoir
travels a long way
before it gets to your house.

But that is not all of its journey.

Some of it has to travel a long long way
before it ever gets to the reservoir.

The water in the reservoir
doesn't all come from the same place.

Some of it comes from one brook
and some from another.

Some of it comes from rain
that rains right down into the reservoir.

Some of it comes from rain
that rains right down into the brooks.

It takes many different little brooks
all hurrying downhill,
over rocks and under bridges.

Many brooks flowing together
and growing bigger and bigger.

It takes the water from many brooks
to keep a reservoir full.

Brooks are always running downhill.

Sometimes the water rushes and tumbles
along.

Sometimes it flows smoothly and quietly.

But it always runs down.

It never runs up
and it seldom stands still.

When water rushes and tumbles along
over mud and sand
and sticks and rocks
it carries some of these things
along with it.

It carries some of the sand
and some of the mud,
bits of sticks and bits of leaves.

The faster the water runs
the more it carries with it.

When water flows slowly
it does not carry so much.

Some of the sand and mud
sink to the bottom.

Some of the sticks and leaves
are left behind.

But when the water runs fast again
it sweeps more things along with it.

It sweeps them right along
to the reservoir.

In the reservoir the water is still
for a while,
stiller than it ever was
when it was in the brook.

So most of the sand and mud
and bits of things
settle down to the bottom
of the reservoir.

They settle down there to stay.

That makes the water much cleaner.

But it does not make it
clean enough to drink.

A lot of things still have to happen
to the water
before it is really clean
and can go to your house.

All day long and all night long
the little brooks bring water
to the reservoir.

But the reservoir is seldom too full.

It seldom overflows.

That is because all day long
and all night long
water is flowing out of the reservoir.

It flows out through a big pipe
to the filtration plant.

The filtration plant is the place
where the water is made very clean.

You cannot tell
just by looking at water
whether it is clean enough to drink.

You cannot tell by tasting water
whether it is safe to drink.

There might be germs in it.

You cannot see the germs
unless you look through a microscope,
and you cannot taste them.

It takes someone
who knows a lot about such things
to find out whether water
is clean enough to drink.

He makes different kinds of tests,
and then he knows
whether it is really clean.

But you do not have to worry about the water
you drink in a city.

The men who take care of those things
see to it that all city water
is very very clean.

In most cities the water goes
to a filtration plant
before it goes to the people's houses.

At the filtration plant
two different kinds of things happen.

One is that certain chemicals
are put into the water.

The other is that the water is made to flow
in certain special places.

All of these things together
make the water clean.

When the water first comes from
the reservoir
it has many little tiny bits
of dirt in it.

It is easier to get out
big pieces of dirt
than it is to get out tiny ones.

So the first thing the men do
at the filtration plant
is to make the dirt in the water
stick together in bunches.

They put a chemical in the water
that does this.

It makes all the little tiny specks of dirt
stick together
to make bigger specks of dirt.

These are called flocs.

If the water is allowed to stand still
some of the flocs settle out of the water.

But other flocs remain.

Now the water is ready to be strained
or filtered.

You remember that when a farmer
strains milk
he pours it through a cloth.

The milk runs through
and the dirt stays on top of the cloth.

The water must be strained too,
but not through a cloth.

The water is strained through filter beds.

A filter bed looks like a big tub.

In it there are three layers,
fine sand on top,
coarse sand below that,
and gravel below that.

The fine sand is the strainer.

The other layers keep the sand in place.

The water runs in at the top
of the filter bed.

It runs down through the fine sand,
down through the coarse sand,
down through the gravel,
and out at the bottom.

The flocs of dirt stay behind.

They get stuck on the grains of sand.

Now the water is much cleaner
but it still is not ready to drink.

It must be aerated.

It flows up into the air
and down again.

In some cities the water bubbles up.

All over a big tank
the water tumbles and rushes
like rapids in a river.

In some cities the water sprays up.

All over a big tank
little fountains of water
spray into the air,
into the air and down again.

When water has been aerated

it tastes better,
and it does not rust the pipes so quickly.

One thing more, before the water
is ready to drink.

It must have chlorine in it,
to kill any germs in the water.

Chlorine is a very strong chemical,
but they put just a little bit
into ever so much water.

Just enough to kill the germs
and not enough to hurt you.

You do not even taste it.

And now the water is really clean, clean
enough to drink.

But the water doesn't go straight
to your house—
not yet.

In most cities it has to be pumped up
to a big standpipe on top of a hill.

That is to get the water so high
that it will flow up the pipes
in your house.

All day long water is being pumped
into the standpipe,
and water is running out of it.

It runs through a big pipe
till it gets to the houses.

It runs through smaller pipes
up into the houses.

The water in the standpipe is higher
than all the buildings in the city.

So the water runs up to the top
of the very tallest buildings
without being pumped.

Down from the brooks
and into the reservoir.

Down from the standpipe
and up in the houses.

All the clean water you want
whenever you want to use it.

SWEET CRISTALL STREAMS OF NEW ENGLAND—THREE CENTURIES AGO

An early tribute to the "cleare running streames" of New England, published only 17 years after the arrival of The Mayflower at Plymouth, "upon tenne years knowledge and experiment of the Country", has just come to my attention. As it appears in a book so rare as to be listed at $375 in a Boston bookshop catalogue of "Poets of America, 1637-1940", which is unlikely to be seen by many water works engineers and superintendents, I pass it on, as quoted in the catalogue:

"Sweete cristall fountains; and cleare running streames,
That twine in fine meanders through the meads,
Making so sweete a murmering noise to heare,
As would even lull the sences with delight a sleepe,
So pleasantly doe they glide upon the pebble stones,
Jetting most jocundly where they doe meete;
And hand in hand runne down to Neptunes Court,
To pay the yearly tribute, which they owe
To him as soveraigne Lord of all the springs.
Contained within the volume of the Land,
Fowles in abundance, Fish in multitude

Which made the Land to mee seeme paradice,
For in mine eie, 'twas Natures Master-Peece:
Her cheifest Magazine of all, where lives her store:
If this Land be not rich, then is the whole world poore."

The lines appear among "several fine passages of poetry (printed as prose) in New English Canaan, by Thomas Morton of Cliffords Inne gent" in 1637.

M. N. BAKER.

Upper Montclair, N. J., March 28, 1941.

TOXIC CONTAMINANTS OF DRINKING WATER†

BY L. T. FAIRHALL*

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Early attempts in the purification of water for drinking purposes were confined largely to improvement of taste and appearance. In effect, they were necessarily crude. For instance, in Ancient Rome, spring water of different degrees of purity or appearance was used for different services. In general, however, river water was supplied to larger cities; and if contamination from industrial wastes was not a serious matter, contamination of other types contributed much to the spread of disease. While man has not lacked a keen appreciation of clean water, it is remarkable that the general water supply should have remained in such a primitive stage of development. Pure water is a distinctly modern achievement and more advance in this field has been made during the past two generations than in all previous history. It is a tremendous tribute to sanitary engineers that this advance has been made in relatively so short a time, that the spread of disease has been so well allayed and controlled, and that clean, acceptable drinking water has been made available in such abundance that even the poorest may feel no restriction on his personal requirements.

In the present day and with newer knowledge, new problems have also arisen, and the sanitarian has a constant struggle to maintain a high standard for drinking water. This relates particularly to toxic contaminants in drinking water. The *natural* contaminants are of less importance. However, contamination from arsenic, selenium or fluorine is occasionally a matter of concern and may arise from water in contact with heavily mineralized formations.

Artificial contamination is another matter. This arises mostly from industrial wastes and the contaminants are of every conceivable variety and description. From food industries, paper and textile mills, tanneries, gas plants, paint and pigment plants, and numerous

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chemical and metallurgical processes, enormous amounts of contaminated water are discharged into rivers and lakes or ultimately find their way there. Mine wastes have contributed not a little of this form of contamination. Until the sealing of abandoned mines was enforced in the bituminous coal fields, 2.7 million tons of sulphuric acid were poured annually into rivers and streams east of the Mississippi River. Prior to the introduction of recovery processes, phenol wastes to the extent of 8,000 to 10,000 tons annually entered the Emscher River in Germany (1).

It has been estimated that an expenditure by public and private agencies of approximately $2,000,000,000 over a period of 10 to 20 years may be required to construct works necessary to abate the more objectionable pollution in this country (2).

The problem of contamination of water supplies is not only related to economic waste, to problems of corrosion of bridges and shipping, or to wild life and fish conservation, but includes above all, the problem of drinking water. Many chemical wastes contain materials injurious to health, and although they are enormously diluted, strict surveillance must be maintained to see that they do not rise to dangerous levels.

In addition to the natural contaminants of drinking water, and to those arising from industrial wastes, one must consider various types of contamination that may arise from piping or from treating the water with softening agents or with anti-corrosion substances. A tendency is noted in respect to the latter, which, however great the economic value, is of questionable nature, where such valiant efforts have been made heretofore to secure a pure drinking water supply.

Of the substances that have at various times or in various localities been reported as contaminants of drinking water, the following are prominent: acids, arsenic, boron, chlorides, certain chromates, various disinfectants, copper, fluorides, saline wastes, iron, lead, manganese, selenium, sodium hexametaphosphate and zinc.

On the other hand, a water may be deficient in certain respects with regard to material necessary for faunal or floral life (dissolved oxygen, for instance)—for these play a significant role in stream life and are of some importance in insuring a clean water. According to Ellis (3), the standards necessary for this purpose are: dissolved oxygen of not less than 5 p.p.m., an acidity range between a pH of

6.5 and 8.5, ionizable salts between 150 and 500 Mho x 10^{-6} at 25° C., ammonia not exceeding 1.5 p.p.m. and suspensoids of a hardness of 1 or greater, so finely divided that they will pass through a 1000-mesh screen and so diluted that the resultant turbidity would not reduce the intensity depth for light penetration to less than 5 meters. Certain streams receiving acid mine waste, as in the Georges Creek coal field area, or contaminated with industrial waste, as in the Shenandoah Valley, or polluted with sewage, as in the case of Rock Creek or the Anacostia River, have been described as biological deserts.

With reference to the undesirable contaminants in drinking water, the question is largely one of degree, and this in turn has been largely determined by practical experience. Standards have been set in a few instances, but this is a difficult and delicate matter, and a quality of water which is easily attained in one area may work an economic hardship in another area. Where a question of health is involved, however, no effort should be spared to correct an undesirable condition.

Arsenic. Although the possibility of arsenical contamination of a water supply (owing to the extensive use of lead arsenate spray material either in orchards for codling moth control, or in grassy areas for controlling the Japanese beetle menace) has been a matter of concern to some, it has not been shown that serious contamination occurrs of a water supply derived from such a water shed.

Analyses were recently made of drinking water in the Wenatchee, Washington district, which is probably the area most heavily exposed to lead arsenate in the United States (4). As much as 7,000,000 lb. of lead arsenate have been used in this area in one year, of the 40,000,000 lb. used annually in the United States. Yet, neither the lead nor the arsenic content of the drinking water supply is seriously affected. For comparison, recent analyses of tap water at the National Institute of Health revealed a lead content of 4 to 7 parts per billion.

Individual instances of contamination of drinking water occur occasionally. Mackenzie (5) cites the case of water (collected from two roofs painted with "pyrites") which contained 1/80 grain per gal. of arsenic and which is stated to have caused several cases of arsenical poisoning. Contamination of well water on a farm, which was due to natural causes, was cited by Wyllie (6) as the cause of illness and one fatality to the occupants of the farm. In this case, the deep well water

LEAD AND ARSENIC CONTENT OF WATER SUPPLIES FROM WENATCHEE, WASHINGTON

Type and Source of Samples	Number of Samples	Lead		Arsenic	
		Average	Range	Average	Range
		Milligrams per Liter			
Drinking Water					
Wenatchee Water Works	20	0.013	0.004-0.050	0.005	0.001-0.012
Wenatchee City Well	3	0.020	0.005-0.032	0.006	0.001-0.013
East Wenatchee Water Works	5	0.015	0.012-0.019	0.006	0.005-0.008
Private Wells & Springs	14	0.005	0.000-0.015	0.005	0.003-0.008
Irrigation Water					
Highline	12	0.016	0.003-0.071	0.006	0.004-0.008
Other	5	0.018	0.006-0.036	0.005	0.003-0.006

was found to contain from 0.4 to 10 p.p.m. of arsenic, as As_2O_3, due to contact with a bed of arsenical pyrites lying within the limestone area. Water from shallow wells nearby was not contaminated. Symptoms of chronic arsenical poisoning appeared after two and a half years' use of the well water. Examination of this water disclosed the presence of minute brown scales or particles which consisted of ferrous arsenate, $Fe_3(AsO_4)_2.6H_2O$.

Contamination of a water supply by arsenic similar to the above has been cited by Stoof and Haase (7) who point out, furthermore, the possible danger of contamination from seepage from smelter waste into surface or ground water. The so-called "Haff sickness" which had been attributed to the arsenic content of the Haff Bay water was probably not due to arsenic, as thought at first, since analysis disclosed an arsenic content of but 5 to 10 parts per billion.

The question of permissible amounts of arsenic in water is largely a matter of opinion with reference to the smaller quantities, whereas direct damage due to concentrations cited by Wyllie is incontrovertible. Stoof and Haase claim that as much as 0.15 p.p.m. of arsenic can be tolerated without injurious effect.

Boron. Boron is restricted in its distribution. It has been reported in drinking water in California and certain districts in Italy. According to Sollmann (8) boric acid and borax behave exactly alike pharmacologically and quantities up to 0.5 grams per day have no immediate effect of any kind on healthy individuals.

Goudy (9) found that the boron content of the Los Angeles

water supply varied from 0.5 to 1.5 p.p.m. He states that this is not harmful in drinking water and that 30 p.p.m. are permissible.

Calcium. Calcium is a dietary necessity, and it is unlikely that drinking water, usable in the ordinary sense, would contain a sufficient amount of calcium to supply the daily need of the average individual. The objection to large amounts of calcium salts is, therefore, not related to health, but is largely an economic problem in that water of this type is "hard", tends to deposit scale upon heating, requires larger amounts of soap than softer waters, and interferes with certain industrial uses unless the calcium is removed by softening agents.

It was found by Heller (10) that a saturated aqueous solution of calcium hydroxide, that is, an 0.18% solution, or 1800 p.p.m., had no apparent ill-effects through four generations of rats, with the exception of a somewhat greater mortality of the young.

If any limit is set for calcium in drinking water, therefore, it is to be set upon a utilitarian rather than a health basis.

Nonetheless, it is of interest that the Spanish Royal Decree of 1920 fixes the limit of calcium in potable waters (as CaO) at 150 p.p.m.

Chromates. Within recent years, a good deal of interest has been taken in possible reduction of corrosion, since the loss per annum from this source is enormous. Various inhibiters, such as sodium silicate and soluble phosphates, have been suggested and tried with the idea of forming a resistor film. One of the means of corrosion inhibition that has met with success is that of film formation with soluble chromates. The surface film thus formed on metals and alloys treated with aqueous solutions of chromates has been such an excellent anticorrosion measure that there has been an increasing application of this procedure. Piping of water coolers has been protected in this manner; the use of soluble chromates in air-scrubbing devices has gradually increased, and efforts have been made to extend this to drinking water supplies, although the protective action of chromates on old piping is open to question.

While the salts of trivalent chromium have been shown to be harmless (11) those of hexavalent chromium have long been known as irritants. In industrial use, they are known to cause diffuse dermatitis, usually on the hands and arms with deep, penetrating ulcers. Furthermore, many cases of idiosyncrasy to minute amounts of chromates have been reported. Apparently, no information is

available with respect to the effects of the ingestion of such small amounts as might conceivably be added to a water supply, but it must be admitted that this would be a step backward were it permitted to occur in drinking water.

With reference to corrosion inhibiters in general, the Editor of the *Journal of the American Water Works Association* recently had this to say (12), "When and if the addition of these substances have been demonstrated to have value in the prevention or control of corrosion, it will be necessary to demonstrate *their safety* as a component of drinking water. Physiological chemists and sanitary engineers are not yet in accord when questioned concerning the addition of some of these materials when there is a possibility that the water may be used for drinking. Therefore, if their value to the metal as corrosion inhibiters is demonstrated, it must not be assumed that their safety to human beings is likewise demonstrated."

Copper. The physiological effect of copper has been somewhat of a controversial subject for a number of years. Mallory (13, 14, 15), in his extensive investigations, has pointed out the relation between chronic copper poisoning and hemochromatosis. The amounts concerned in bringing about this condition are, however, much greater than those likely to be found in drinking water.

Whether or not copper is toxic to human beings is still unestablished. The literature is replete with regard to its harmlessness to the human organism. On the other hand, one repeatedly encounters references to chronic copper poisoning. It has been stated (16) that, in the long run, copper causes gastrointestinal catarrh. However, the susceptibility varies very much for different individuals, and it is probable that the human organism gradually adapts itself to the quantity of copper taken daily.

It should be mentioned that a new physiological function for copper has been elucidated by more recent researches on experimental anemias. A few years ago, it was shown that the milk anemia, which could be produced in rats even with a supplementary iron ration, was not produced in those animals receiving a supplementary ration of both iron and copper.

Except in cases where overdosage of water with copper sulphate occurs through accident or carelessness, or where a very acid water is

conveyed in brass or copper piping, the copper content of water runs quite low. Even in the latter case, the amount of copper in the drinking water is so very much lower than the amount of copper ingested as a normal constituent of food that it has no practical health significance.

Fluorides. Traces of fluorides are found in many drinking waters; in heavily mineralized regions, the fluoride content may reach high levels, and in view of the investigations of H. T. Dean (17) and others, the fluoride content of drinking water, in general, has become a matter of increasing public interest. A relationship between the mottling of teeth, or dental fluorosis, and fluoride content of drinking water has become definitely established. The most recent study of dental fluorosis by McClure (18) has revealed the limits within which definite damage may be likely to occur. According to McClure, acute toxic effects of sodium fluoride in drinking water occur at a level of about 180 p.p.m. of fluorine. Retention probably occurs in the human when fluorine is present to the extent of 2 to 3 p.p.m. of fluorine and this retention may be increased by the presence of more than 2 to 3 p.p.m. of fluorine.

Further work which confirms this figure has also been reported by Trelles (19), who bases his results on dental alterations apparent in people using waters of various fluoride contents. There were severe dental alterations in groups using water of from 2 to 5 p.p.m. of fluorine; at a level of 1.2 p.p.m. of fluorine, slight alteration in tooth enamel was evident, but no alterations were found among the population using drinking water containing fluorine up to 0.6 p.p.m. It is of interest in this connection that recent epidemiological evidence points to an *inverse* relationship between endemic dental fluorosis (mottled enamel) and dental caries (20, 21).

Fluorides are, in general, natural constituents of drinking water, except in a very few cases of factory waste contamination from cryolite plants. They should be made a matter of inquiry, however, in assessing the purity in any water supply.

Lead. With few exceptions, such as contamination from industrial wastes, or from a heavily mineralized water shed, the presence of lead in drinking water, except in traces, is unusual. Most of the lead contamination of drinking water that ordinarily occurs is probably due to the improper use of lead piping. Many cities use lead services, since

this material meets conditions such as resistance to corrosion and vibration, and (since it is a poor conductor) it is less vulnerable to electrolytic corrosion. According to Ziegfeld (22), there is little, if any, danger from lead piping used for public supplies in the United States outside New England.

However, changes in the character of a water may produce unforeseen results, as in the July 1930 epidemic of lead poisoning in Leipsig. Kruse and Fischer (23) showed a relationship there between lead poisoning and water consumption. Samples of water taken early in the morning, where the water had laid in the pipes, contained 5.3 p.p.m. of lead, and the danger limit was shown to be 2 p.p.m. Beneficial results were obtained by treating this water with lime.

The accelerated corrosion of lead by aerated soft water was shown years ago by Weston (24) as well as the protective action of temporary hardness, and the permissible figure of 0.1 p.p.m. of lead established by him is still incorporated in our water standards. This is a lower amount than that permitted either in Holland (25) or in Germany (26) where 0.3 p.p.m. is considered permissible.

It should be pointed out that water in the absence of lead piping is occasionally contaminated with lead. This is due to the careless use of white lead plumbing compound at the joints. The excess plumbing compound should be well flushed out of recent plumbing, and in any case, it is always advisable to draw off water that has lain in pipes before water is used.

Manganese. Manganese salts, when administered by mouth, produce no noticeable effect except in large amounts or when in the form of manganates or permanganates. In the body, it appears to take much the same course as iron and in fact appears to be an essential substance, since it is found in most plants and in all the organs in man and animals. The average daily intake in man is about 10 milligrams —derived from his food. Manganese poisoning occurs only from the inhalation of manganese-containing dusts. While water containing excessive quantities of manganese causes brown stains in cloth in the textile industries, is said to have a detrimental effect in fermentation (27), and causes deposition of manganese oxide in piping and on filter beds, it does not seem to have any harmful effect upon man and would seem to be no more objectionable in this regard, than iron.

Hexametaphosphates. There has been some discussion in recent

years with reference to the addition of sodium hexametaphosphate to water for softening purposes. Its properties in this respect are remarkable. Whether there is any health risk in adding this substance to drinking water is difficult to say, as little is known concerning the physiological action of sodium hexametaphosphate. According to Cushny (28), the phosphates are inactive after absorption. If it is true, as has been stated (29), that sodium hexametaphosphate is broken down in the stomach to sodium phosphate, it would then appear that there is little need for agitation over the problem from this point of view. However, it would seem preferable that a searching investigation of the physiological effects of small amounts of sodium hexametaphosphate be undertaken as well as its effects on the supply before it is permitted to be added to drinking water in amounts exceeding a few parts per million.*

Selenium. A few years ago, the relationship between the so-called "alkali disease" in cattle and the selenium content of cereal grain in certain sections of South Dakota was demonstrated (30, 31, 32), and some concern has been expressed regarding the possibility of selenium contamination of drinking water, particularly in water draining from a seleniferous water shed.

Little is known of selenium poisoning in man. Dudley (33) has evaluated the selenium absorption of employees in a copper refinery, but most of our knowledge of selenium poisoning relates to the poisoning of livestock through eating seleniferous grain or herbage.

The amounts of selenium occurring naturally in water even in seleniferous regions appear to be quite low.

Byers (34) gives the selenium content of water from the Colorado River Basin as ranging from 0 to 400 parts per billion. Strock (35) reports 3 to 4 parts per billion in the waters of the North Sea. Miller and Byers (36) investigated a so-called "poison spring" which was found to contain 0.4 to 0.5 p.p.m. of selenium but this was stated to be too low in amount to cause cattle poisoning.

Some substances such as zinc compounds and certain industrial wastes are indicated in drinking water by an objectionable taste before the amounts present are great enough to be of hygienic importance and in this way are self limiting. Taste, alone, however, is not a

*Editor's Note: According to K. K. Jones—"The Physiology of Sodium Hexametaphosphate" *Jour. Am. Water Wks. Assn.* 32, 1471 (1940)—hexametaphosphate is no more toxic than any physiologically active salt.

criterion in most cases, and the most careful control of a drinking water supply is necessary at all times. Fortunately, in the case of *natural* contaminants, evaluation of the constituents of possible harmful toxicity is needed only at rare intervals. In the case of contamination from *industrial* wastes, the sanitary engineer is liable to be faced with an acute situation requiring immediate solution.

The fact that drinking water in general has attained such a high standard of purity is a technical achievement of high order and a tribute to the efficiency and idealism of the sanitary engineer.

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DETERMINATION OF COMPARATIVE EFFICIENCIES OF FILTER WASHING

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INTRODUCTION

In the purification of water for drinking purposes, filtration plays a major role. Slow sand filters are used to a great extent for waters comparatively low in color and turbidity that are not subject to excessive pollution, while rapid sand filters are more practical for surface waters high in color and turbidity. All filters must be cleaned after being in service for a certain length of time. In the slow sand type, the filter is cleaned by scraping a thin layer of sand off the top. The filter is then ready for use again. The sand that has been removed may be washed outside of the bed and replaced when necessary. Rapid sand filters, on the other hand, go through a more elaborate process of cleaning, the sand being washed in place. Older types of rapid sand filters employ devices such as rakes for stirring up the sand, the wash water carrying off the accumulated dirt. Later developments make use of low-velocity washing with the aid of air for cleaning the sand grains. The newer methods employ high velocity washing, ordinarily without the use of air.

In discussing methods for performing a certain job, the question generally arises as to what method will perform the necessary work most efficiently. This question also suggests itself in connection with filter washing. Which of the three methods of rapid sand filter washing just mentioned is the most efficient, or what are the comparative efficiencies of these three methods? In order to find an answer to this question, it is first necessary to develop a suitable method for measuring the dirtiness of sand filters prior to and subsequent to washing.

This paper deals with a study of methods for determining the

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dirtiness of sand filters and the application of such methods to a first study of comparative efficiencies of the three common methods of rapid sand filter washing.

Filter Plants Included in This Study. The rapid sand filter plants studied were the East Providence and Warren (Bristol County Water Co.) plants in Rhode Island, and the Cambridge plant in Massachusetts. Each of these plants employs a method of filter washing different from that in use at either of the other two plants. The East Providence filter plant employs the oldest form of filter washing. The filters are of the circular, wooden type, and the method of washing is low-velocity backwash and the use of rakes. A vertical rise of 12 in. per min. is obtained, the washing period being 10 to 12 min. The filters are 15 ft. in diameter and have a sand depth of 29 in. The retention period in the coagulation basin, with all six filters in service, is three hours. The source of supply is a river water.

At the Warren plant, there are six rectangular filters, concrete lined, the sand being cleaned by means of low-velocity washing accompanied by the use of air. The air is applied for three minutes, and the filter is then washed for five minutes, the vertical rise being about 15 in. per min. The filters are 12 ft. by 15 ft. in plan and have a sand depth of 22 in. The retention period in the coagulation basin, with all filters in service, is 45 minutes. The source of supply is an impounded water that is stored in a pond for a short period.

The Cambridge plant has 16 rectangular filters that are cleaned by high-velocity washing without air. A vertical rise of 22 to 28 in. per min. is obtained, and the filters are washed for 2 to 3 min. These filters are 20 ft. by 24 ft. in plan and have a sand depth of 25 in. The retention period in the coagulation basin is about 2.1 hours with all filters in use. The source of supply is an impounded water having a rather long period of storage when compared with the waters of the other two plants.

Methods Used in This Study

The main object of this study was to devise a reliable method for determining the dirtiness of filter sand. Use could then be made of this method to find the amount of dirt in a filter at the end of the run and after washing. Having these values, the difference would

give the amount of dirt removed from the filter by the washing process. By dividing this value by the amount of dirt imparted to the filter by the water filtered through it, we should be able to obtain the efficiency of washing.

In order to obtain samples of sand on which to perform the various tests to be described later, the filters were drained to the bottom of the sand bed. A one-inch brass pipe was used to obtain samples. This pipe was marked off into divisions, so that samples could be taken to represent various depths in the filter. By boring into the sand up to the first mark, the top inch of sand was obtained. This sample was blown into one of the sampling bottles. The pipe was then bored into the same hole in the sand up to the second mark which was two inches from the first one, thus giving a sample for the next two inches of filter depth. Samples were collected in this manner at different levels in the sand bed until the bottom of the bed was reached, each sample being placed into one of the sampling bottles properly labeled. The sampling bottles used were 2-ounce, wide-mouthed bottles with screw caps, capable of holding a sample of sand collected by a one-inch pipe to a depth of four inches. Samples were taken from different sections of the filter to obtain representative conditions, the corresponding samples for the different depths being composited. The tests were performed on the composited samples.

Samples of sand were collected at the various filter depths at the end of the filter run. The filter was then washed, and samples were again collected in the same locations. By this method of sampling, it was hoped to determine the amount of dirt accumulated in the filter during the particular run for which the samples were collected. The differences between the expressions for the amount of dirt for corresponding samples collected at the end of the run and after washing should give the amount of dirt removed from the different layers of sand by washing.

Before performing the actual work of measuring the dirtiness of sand filters, sieve analyses were made of the sand used in the filters of the three plants to determine the sand characteristics. The usual methods employed in sanitary laboratories were used for this purpose. A sieve analysis was made on a composited sample for the entire filter depth for each plant, as well as on a sample of the top inch of sand for the Cambridge plant and on the top two inches for the East

Providence and Warren plants. The results of the mechanical analyses of these samples of sand indicated that the East Providence and Warren sands were very similar and practically uniform throughout the entire depth of the filter, the effective sizes for the top two inches being 0.58 and 0.615 mm., respectively, while for the composited samples for the entire filter depth these values are 0.59 and 0.620 mm.

The effective size of the sand from the top inch of the Cambridge filter, however, was much smaller than that for the composited sample for the total depth of the bed, the former being 0.31 mm. while the latter was 0.50 mm.; that is, the Cambridge sand was graded from a fine sand in the top layer of the bed to coarser sizes in the lower sections. Also, the top layers of the East Providence and Warren filters were much coarser than those of the Cambridge filters.

Early in this study experiments were performed on four possible methods for measuring the dirtiness of filter sands: (1) the sodium hydroxide color test, (2) the sulfuric acid color test, (3) the determination of the loss on ignition and (4) the alum test. The reasons for choosing these methods were as follows:

1. Sodium Hydroxide Color Test. Sodium hydroxide is used in many filter plants to remove dirt that accumulates on the grains of sand over a long period of time. Generally, a solution is applied to the sand and allowed to remain in contact with it for 24 to 48 or more hours. This treatment dissolves the dirt that has accumulated on the sand, and a highly colored brownish solution is the result. By washing out this brownish sodium hydroxide solution after the desired period of contact, the dirt is removed from the filter, whereupon it is ready to be put back into use.

This practice was adapted to the present study by treating samples of sand from the beds of the three plants with a 3% solution of sodium hydroxide and comparing the colors produced thereby with standards. It was found that, upon diluting the colored sodium hydroxide solutions resulting from treating the sand with caustic, the colors could be compared with the ordinary color standards used in water purification plants. The manner in which this test was applied to the determination of the dirtiness of sand filters is explained later in this paper.

2. Sulfuric Acid Color Test. It was found that when representative portions of the same samples of sand that had been treated

with caustic were treated with a 3% solution of sulfuric acid, the colors obtained were but ⅓ to ½ as high. Figure 1 illustrates the comparative amounts of color removed from the samples of sand by the two solutions. It is apparent that the sodium hydroxide solution extracts a much greater amount of dirt from the grains of sand than does the sulfuric acid. The optimum method for determining the dirtiness is that which measures as much of the dirt as possible. On this basis, the sodium hydroxide method is the one to be preferred. The sulfuric acid method was therefore discarded and was not used after this preliminary experimental work.

3. Determination of Loss on Ignition. This test was performed on samples of sand by first drying them, weighing, igniting and then weighing again to determine the loss on ignition as a measure of organic dirt. Since the results obtained on one set of samples was not promising and the test was time-consuming, it was abandoned.

4. Alum Test. Inasmuch as all three filter plants studied used alum as a coagulant, the fourth method studied was Hatfield's* modified hematoxylin test for alum, on the premise that determination of the alum going over into the filters would provide an index of sand dirtiness. Preliminary trials indicated that this test might prove valuable.

Sodium Hydroxide Color Test

The sodium hydroxide color test was performed by placing samples of sand in 8-oz. prescription bottles, adding a known volume of sodium hydroxide solution, stoppering the bottles and shaking them a definite number of times. The treated samples were stored for at least 24 hours and inverted several times to obtain a homogeneous color through the entire solution; then measured portions ranging from less than one to five cubic centimeters of the solution were pipetted into 50-cc. Nessler tubes for dilution with distilled water for readings against color standards made in accordance with the methods outlined in "Standard Methods of Water Analysis."

When these tests were first started, the sand was dried and portions of 10 grams were placed in the bottles; 50 cc. of 3% sodium hydroxide solution were then added and the bottles shaken 25 times

*Hatfield, William D. Soluble aluminum and the hematoxylin test in filtered waters. Ind. Eng. Chem., Vol. 16, 233 (1924).

before storing. All results were expressed as color in parts per million per gram of sand.

In later tests, however, a modification was made. Instead of drying the sand prior to placing it in the bottles for treatment, the tests were performed on the sand in its original wet state as obtained from the filter, since it was found by experiment that by drying the samples of sand lower alum readings were obtained. By using sand in a wet state, it was found that a fixed weight of sand could not be used for the individual samples from different depths, since these samples contained varying amounts of moisture. It was found necessary, therefore, to get approximately equal weights of sand by measuring equal volumes of sand, applying the caustic, obtaining the total color results, filtering and drying the sand and then weighing it. The total color results were then divided by the corresponding weights of dry sand to give the results as color per gram of sand.

The quantity of sodium hydroxide solution was also changed from 50 to 200 cc. As shown in Figure 1, it was found that, practically speaking, varying the quantity of sodium hydroxide affects the color results only to the extent of the dilution introduced by the additional amount of sodium hydroxide solution applied to the sand. However, it was thought better practice to add a larger quantity of sodium hydroxide solution to the sand so that the dilution factor later employed in diluting the samples to be tested for color would not be as high as when only 50 cc. of the caustic solution were added to the samples. Also, by using only 50 cc. of caustic, quantities often less than 1 cc. of the highly colored solution had to be removed from the bottles and diluted to 50 cc. in order that the colors might fall within the range of the standards. This might introduce large errors due to the high dilution factor. Furthermore, the errors in measuring such small quantities by use of ordinary laboratory pipettes were obviated by changing to the use of 200 cc. of caustic solution.

The sodium hydroxide used in bringing out the color from the sand was a 3% solution. It was decided to use this strength since, in filter plants where sodium hydroxide solution is used for cleaning the sand, the strength usually employed is from 2 to 5%, and also since 3% sodium hydroxide is used by highway laboratories in determining the organic impurities in sands.

There is no special reason for shaking the bottles containing the

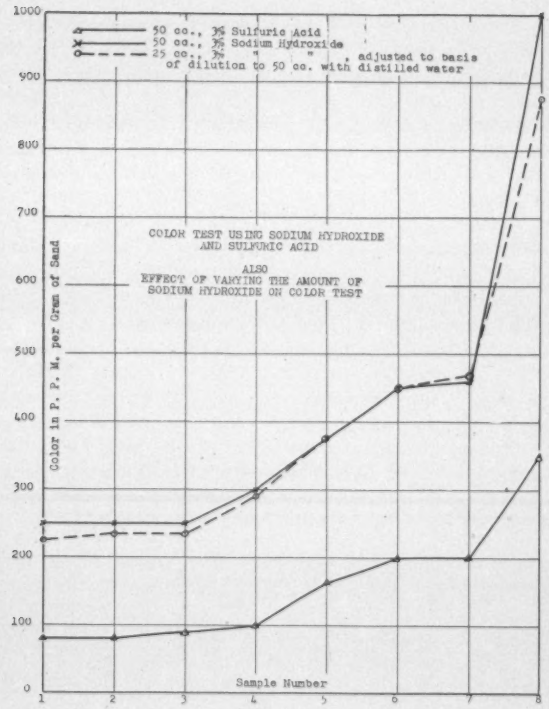

FIGURE 1.

sand and caustic 25 times. This is an arbitrarily chosen value, it being thought that this amount of shaking should be sufficient to dissolve the organic impurities from the grains of sand. As is shown later, longer shaking does not increase the amount of color produced.

ALUM TEST

In the preliminary test made for the determination of the alum content in the dirt coating on the sand, it was found that by taking

from 2 to 5 cc. of the diluted color solutions from the Nessler tubes, after having read the colors and inverted the tubes a few times to insure a homogeneous solution, and diluting these quantities to 50 cc. in Nessler tubes, alum tests could be performed by the Hatfield modification of the hematoxylin test. By using this procedure, the alum results obtained were found to fall within the range of alum standards convenient for use, namely from 0.1 to 1.0 p.p.m. of alum as Al_2.

Effects of Various Conditions on the Tests

Anyone who has attempted to do research realizes that during the early part of such work, and sometimes during the entire research period, questions come up as to the effect of one or more conditions on the problem as a whole. The various questions that arose as time went on during this work will now be discussed.

Effect of Drying the Samples of Sand. As stated before, it was found that drying the samples of sand prior to adding the caustic for the color and alum tests had a tendency to reduce the alum readings, probably due to a change in the form of the alum. On the other hand the color readings had a tendency to be increased. Figures 2

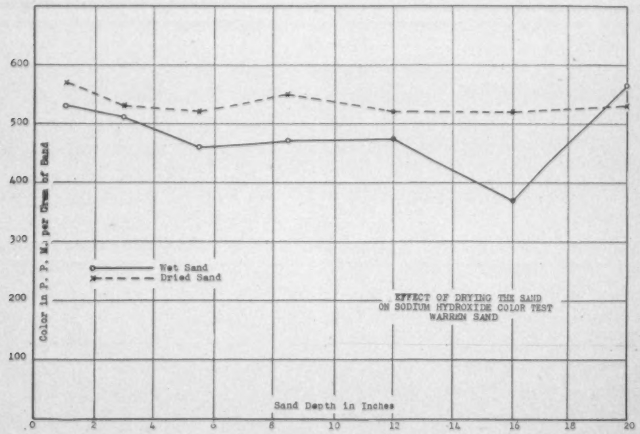

FIGURE 2.

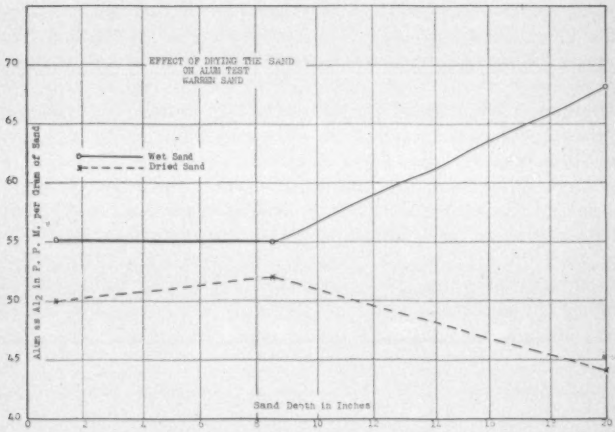

FIGURE 3.

and 3 show the effects that the drying of the samples of sand had on the color and alum tests.

Effect of Varying the Amounts of Sand Used. The next effect to be investigated was that of varying the amounts of sand used in these tests, employing a constant volume of caustic solution. In the first tests, the samples of sand were dried, and equal representative portions were used, that is, a 10 gm. portion of each. In later tests, however, the sand was used in its wet state, so that samples of equal weight could not be obtained due to variations in the moisture content of the sand and other factors. It was necessary, therefore, to resort to a method of obtaining representative samples that would be of approximately equal weight for the samples from the different depths of the filter. Representative samples of approximately equal volumes were therefore taken and, after having added the caustic and obtained the color and alum readings, the sand was removed from the respective bottles, dried and then weighed. It was found that by using this technique the weights of sand varied from 6 to 16 grams. The question arose as to whether the relative amounts of dirt re-

moved per gram of sand would be comparable for different weights of sand and equal volumes of sodium hydroxide. Tests were therefore performed to determine the effect of varying the weight of sand on the color and alum tests. In these tests, varying amounts of the same sample of sand were taken, 200 cc. of sodium hydroxide solution were applied to each, and the color and alum readings were made.

The results indicate that the color and alum readings are affected by varying the weight of sand used. That is, for constant volumes of sodium hydroxide solution and varying weights of sand, increases in the amount of sand tend to decrease the color and alum readings per gram of sand. It was necessary, therefore, to keep the weights of sand used as nearly alike as possible. Figure 4 shows the effects of varying the weight of sand.

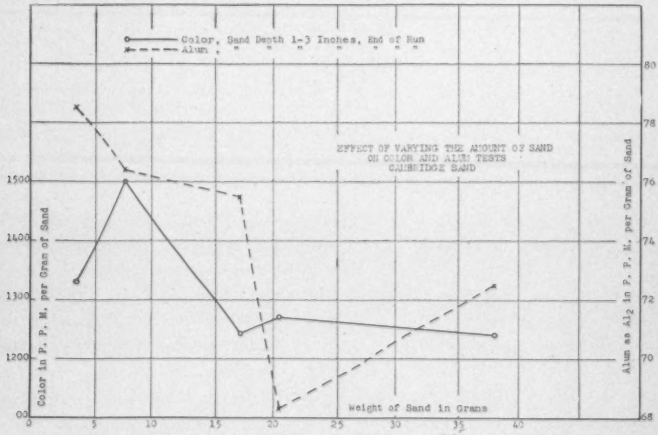

FIGURE 4.

Effect of Varying the Strength of the Sodium Hydroxide Solution. After having treated the samples of sand from the different plants with sodium hydroxide solution for the determination of color and alum, and after having poured off the supernatant colored caustic from the bottles containing the sands, it was noted that while the

East Providence and Warren sands had practically all of the organic impurities removed from the sand grains as judged from their clean appearance, the Cambridge sand looked as though it had not given up all of its organic matter. In its dirty state prior to treatment with caustic, it had a reddish-brown appearance, and after the caustic treatment, it still had somewhat the same appearance. The question arose as to whether or not all of the dirt was being removed by using the 3% sodium hydroxide solution. Experiments with varying strengths of sodium hydroxide solution were therefore performed to

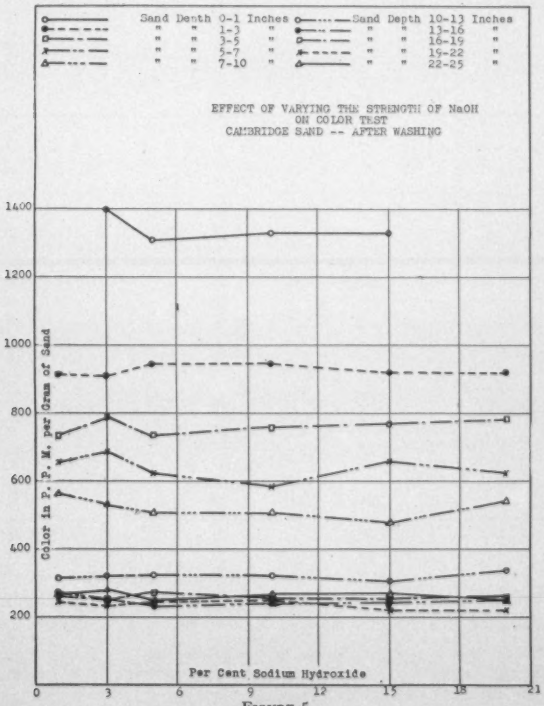

FIGURE 5.

determine the optimum strength to be used for the removal of the maximum amount of organic matter. For samples of sand from various depths of the Cambridge filter, it was found that varying the strength of the sodium hydroxide solution from 1% to 20% did not change the color readings materially, indicating that strong solutions of caustic do not remove more dirt per gram of sand than does the 3% solution. The alum readings, however, were not consistent in

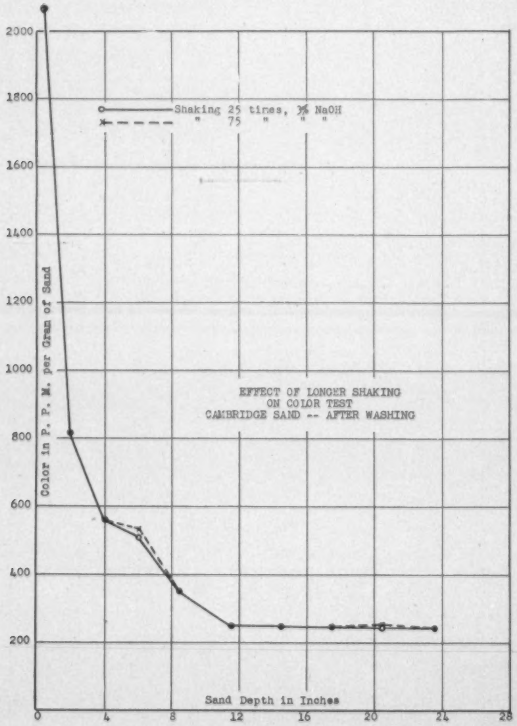

FIGURE 6.

their behavior for different strengths of caustic, one set of data showing an increase in alum readings for increased strengths of caustic, while another set had a tendency to have decreasing amounts of alum extracted by increased strengths of caustic. Figure 5 shows the effect of varying the strength of caustic on the color readings per gram of sand.

Effect of Longer Shaking of Samples with Caustic. Since it was found that after treating the Cambridge sand with stronger solutions

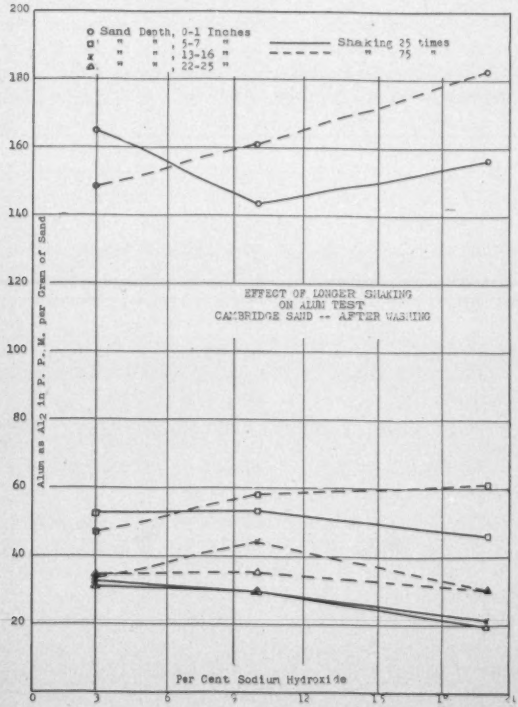

FIGURE 7.

of sodium hydroxide the sand still maintained a reddish-brown appearance and did not give up any more color as explained before, it was thought that a more effective cleaning might be obtained if the bottles containing the sand and caustic were shaken more than in previous tests, namely 25 times. Bottles that had been shaken 25 times were, therefore, shaken 50 times more. The results indicate that there is absolutely no change in the color readings due to an increased amount of shaking. The alum results, however, seem to be increased when samples are given a longer shaking with caustic. Figures 6 and 7 show the effects of longer shaking on the color and alum tests.

From the results reported, it is apparent that neither the use of strong solutions of caustic nor long shaking was effective in giving the Cambridge sand a clean appearance. However, the residual color-producing material on this sand is of insignificant magnitude when compared with the quantity of dirt removed from the sand by the caustic. Also, the samples of sand collected at the end of the filter run and after washing the filter would have equivalent amounts of residual dirt not removable by the caustic. For these reasons it was not regarded as very likely that the reddish-brown appearance of the Cambridge sand after treatment with caustic would introduce any serious errors into the results obtained by the color test.

Comparative Values of the Sodium Hydroxide and Alum Tests

As a result of a study of the data accumulated during this research, especially the data pertaining to the effects of the different variables that have been discussed, it was found that the alum results are affected by every one of the variables, in some cases materially. It should also be noted that the alum test is a very delicate one, in that the slightest variation in the procedure introduces large errors. It is also a fact that but a few alum tests can be made at a time, since the colors produced are not stable and do not remain the same for any appreciable length of time, therefore necessitating the preparation of new alum standards for each set of alum readings. In some of the work, it was found necessary to make as many as three sets of alum standards in one day. For such reasons, the alum test was not considered worth while as a means for determining the relative amounts of dirt contained in the different samples of sand.

The color test, on the other hand, gave results that were fairly

consistent throughout the work done on this problem. These results are comparable for the samples from the different depths of the filter beds, as well as for samples from the three different plants, as is evidenced by the consistent results obtained at the different times of sampling.

Summary of Results

Having arrived at the most satisfactory determination of filter dirtiness, namely the 3% sodium hydroxide color test, the data obtained by means of this method were used to estimate the amounts of dirt removed from sand by washing. The averaged colors for the samples of sand from each of the various filter depths were determined for each plant. These averaged colors were plotted against filter depth on double logarithmic paper, and the method of least squares was employed in determining the equations of the lines of best fit shown in Figure 8.

The differences between the average colors for the samples col-

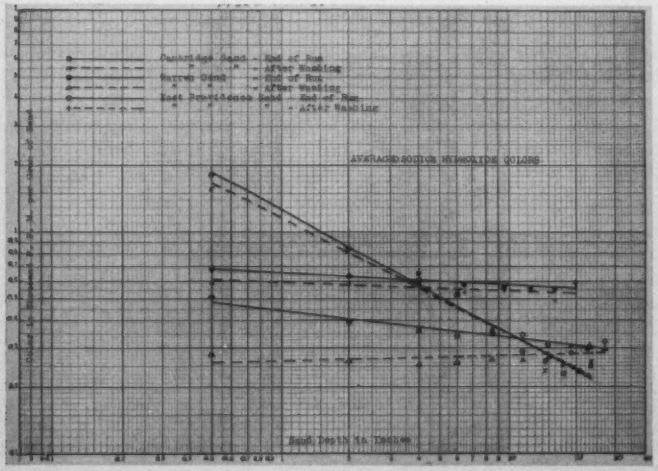
Dunbridge Sand - End of Run
" " - After Washing
Warren Sand " - End of Run
" " - After Washing
East Providence Sand - End of Run
" " - After Washing
AVERAGED SODIUM HYDROXIDE COLORS
Sand Depth in Inches

Figure 8.

lected at the end of the filter run and after washing the filters were then obtained. Thus, the amount of dirt removed per gram of sand was determined for the various filter depths for each plant. From these values the percentage of dirt removed by the various layers of the filter bed was obtained, as shown in Figure 9. It will be noted

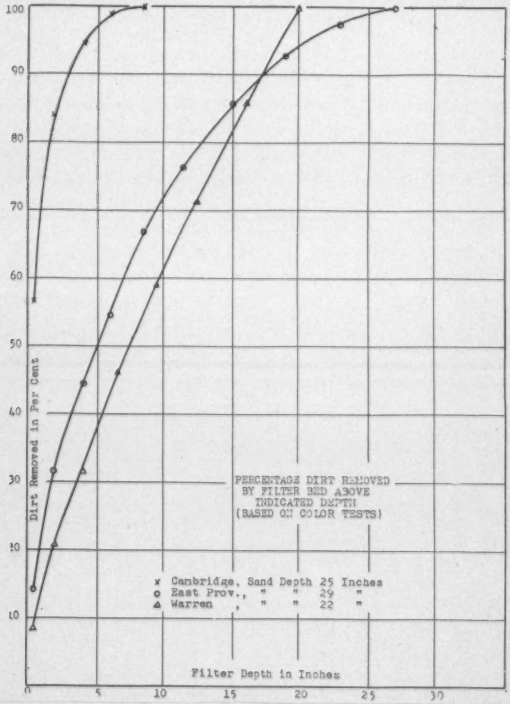

FIGURE 9.

that for the Cambridge filter practically all of the dirt is removed by the top few inches of sand; 95% of the dirt is removed by the top 5 inches of sand; in fact, the top inch removes 57% of the dirt. As

was explained at the beginning of this paper, the top layer of the Cambridge filter consisted of very fine grains of sand, while the lower layers were graded to a coarse sand at the bottom. Therefore, in filters such as those used at Cambridge, the total effective filtering area of the grains in the top layers of sand is greater than that for a bed with coarse grains in the top inches of the bed. Hence, there is little chance for dirt to penetrate much beyond the first few inches of sand.

For the East Providence and Warren plants, a depth of 21 and 18 in., respectively, are utilized to remove 95% of the dirt, the top inch at these plants removing but 14 and 9% of the dirt, respectively. While the top inch or two at these two plants removes a fair portion of the dirt, there is also considerable removal by the lower layers of sand in the filter.

To determine the efficiency of filter washing from the results of the sodium hydroxide color test, it is but necessary to determine the total amount of dirt applied to the filter by the water passing through it and that which is washed out from the filter. Dividing the latter by the former, the percentage of dirt removed by washing or the efficiency of washing is obtained. To determine the amount of dirt imparted to the filter by the water being filtered, color readings must be made on the filtered and applied water. The color on the latter should be read after adding sodium hydroxide solution to it, in order to have these results comparable with the colors obtained by shaking the sand with caustic. By reading the colors on the samples of applied water without the addition of sodium hydroxide, the colors are considerably lower and would correspond to colors produced by shaking sand with distilled water. The difference between the colors of the applied and filtered water gives the amount of color removed from 50 cc. of water. This value multiplied by 75.7 gives the color removed from one gallon of water. Knowing the length of filter run in hours and the rate of filtration in gallons per minute, the total number of gallons of water filtered during the run is obtained. The total amount of color imparted to the sand by the water filtered through it is obtained by multiplying the color imparted by one gallon of water by the number of gallons filtered.

The amount of color removed from the filter by washing was obtained as follows: First the weighted averages of the color per gram

of sand were obtained for the samples collected after washing and at the end of the run. These values were obtained by multiplying the color per gram of sand for the top inch by one, that for the next two inches by two and so on for the remainder of the depth of the filter. The sum of these products was divided by the depth of sand in the filter, thus giving a weighted average per gram of sand. The difference between the weighted average color values for the samples collected at the end of the run and after washing the filter gives the average amount of dirt washed out of the filter per gram of sand. Multiplying this average color value by the total weight of sand in the filter, the total amount of color washed out of the filter is obtained.

Thus, having obtained an expression for the amount of dirt applied to the filter and that washed out of the filter, it is possible to determine the efficiency of filter washing. This procedure was followed for the few sets of samples that had been tested, and it was found that the East Providence plant, where antiquated rakes are employed to assist in the removal of dirt, appeared to possess a more efficient

COMPUTATION OF EFFICIENCY OF FILTER WASHING

	Cambridge	*East Providence*	*Warren*
Color removed from filter by washing			
Dimensions of filter—ft.	20x24	15 (diam.)	12x15
Area of filter—sq. ft.	480	177	180
Sand Depth—ft.	2.08	2.42	1.83
Volume of sand—cu. ft.	1,000	428	330
Volume of sand—cu. m.	28.3	12.1	9.3
Specific gravity of sand (assumed)	2.65	2.65	2.65
Weight of sand per cu. m.—gm.	2.65×10^6	2.65×10^6	2.65×10^6
Weight of sand in filter—gm.	75×10^6	32×10^6	25×10^6
Color removed per gm. of sand by washing—p.p.m.	7	60	6
Color removed from filter by washing—p.p.m.	0.53×10^9	1.92×10^9	0.15×10^9
Color removed from water by filter			
Color removed from 50 cc. of water—p.p.m.	35	43	13
Color removed from 1 gal. of water—p.p.m.	2,650	3,255	984
Rate of filtration—gal. per min.	1,036	347	347
Filter run—hr.	9.9	29.5	9.25
Gallons filtered	615×10^3	614×10^3	193×10^3
Color removed from water—p.p.m.	1.63×10^9	2.0×10^9	0.19×10^9
Efficiency of filter washing—%	32	96	79
Efficiency of filter washing (av.)—%	32	84	75

method of filter washing than the other plants. The accompanying table shows typical computations of percentage efficiencies of washing.

It should be noted, however, that these findings are based on the results of but a very few samples. Therefore, any definite conclusions drawn from these results would have little statistical significance. A definite conclusion that any plant has a more efficient method of washing than any other plant cannot be drawn until a significant number of samples has been tested. However, the material reported in this paper establishes the fact that the sodium hydroxide color test is an effective one for determining the relative efficiencies of various methods of filter washing.

Conclusions

1. Four methods were investigated to determine an index of filter dirtiness. Two of these methods were discarded for reasons mentioned in the paper and the other two were selected for more intensive study. Finally, one of these methods, namely the sodium hydroxide color test, was found to be satisfactory.

2. The findings as to the sodium hydroxide color test are: (a) the color produced by shaking samples of sand with sodium hydroxide is a good means for determining the relative amounts of dirt removed from sand by various methods of filter washing; (b) in view of the fact that it is practically impossible to obtain samples of sand of equal weight when using sand in the wet state, the sand should be dried and representative portions of the dried sand should be taken prior to making the color tests; (c) by varying the strength of sodium hydroxide from one to 20 per cent there is no appreciable change in the amount of color removed per gram of sand; (d) there is no need for shaking the bottles containing the samples of sand and the caustic more than 25 times, since the color test is not affected by longer shaking; and (e) sodium hydroxide should be added to the samples of applied water prior to reading the colors on these samples, since by following this procedure we obtain a color corresponding to the color produced when caustic is applied to the sand.

3. The present study deals with methods for determining filter dirtiness rather than with statistically significant comparisons of the three types of rapid sand filter washing. However, the old method of low velocity washing, aided by the use of rakes, showed some indica-

tion of being more efficient than the other two methods of washing studied. With the method now available, more extensive data are needed to draw final conclusions on the relative efficiencies of various methods of filter washing.

Acknowledgment

The author should like to acknowledge gratefully the abundant and very helpful suggestions of Professor Gordon M. Fair during the course of this study which was initiated under his guidance. The author also acknowledges his gratitude to Mr. Charles L. Pool, Chief, Division of Sanitary Engineering, Rhode Island Department of Health, for the valued comments made during his review of the material contained in this paper.

MILITARY FIELD SANITATION*

BY W. A. HARDENBERGH†

[*Read April 3, 1941.*]

The objectives of military sanitation are the prevention or control of disease and the maintenance and conservation of the health of the Army. These objectives are attained by applying the same general principles of sanitation that are employed in civilian health work; but because of the military environment, the details of application of these principles may differ. Measures of military sanitation must always be adapted to the situation and to the mission of the command.

Commanding officers of all grades are responsible for sanitation and for the enforcement of sanitary regulations and orders within their organizations and inside the boundaries of areas occupied by troops under their command. Full use should be, and normally is, made of the technical advice and knowledge of the Medical Department officers, but the Commanding Officer has the full responsibility for the initiation and enforcement of suitable measures for the correction of sanitary defects and for safeguarding health.

The Medical Department is responsible for investigating, reporting on and making recommendations relative to all matters affecting the health of the Army, including the location of camps, the sources of water supply and the methods of water purification; the methods and efficiency of waste disposal; the quality of the food and the sanitation of messes; the suitability of clothing and housing; the efficiency of training in personal hygiene and sanitation; the control of insects; and all other measures having to do with the prevention or control of diseases. The Medical Department is also responsible for the execution of health measures requiring technical skill, such as physical inspections, immunizations, food inspections, planning and supervision of insect control, and similar work. The Medical Department does not furnish labor, funds or equipment for such activities as water purification, waste disposal or insect control.

*A paper read before a joint meeting of the New England Health Institute, the New England Water Works Association and the New England Sewage Works Association.

†Colonel, Sanitary Corps, U. S. Army.

Thus the Medical Department functions are mainly those of inspection and advice. Medical Department officers do not exercise command except in Medical Department organizations. However, their recommendations constitute the basis for initiation of proper sanitary measures by the Commanding Officer. To be most effective, these recommendations should be "pertinent" and "timely," that is, such recommendations must be made in time for effective action, but not too far in advance of actual need; and they must be practical and effective when judged by the effort required, the results obtained and the military mission of the troops.

The senior medical officer of a command is called the Surgeon. He is charged with the general supervision of all Medical Department activities of that command. The Medical Inspector is an assistant to the Surgeon and, under his general direction, is charged with supervision over sanitation. In large camps or stations, or in the field, the Medical Inspector may have one or more assistants; and in such cases, these assistants are often specialists such as sanitary engineers, sanitarians or entomologists.

Since the Medical Department does not furnish labor, funds, or equipment for such work as water supply, waste disposal, or mosquito control, other branches of the service have functions and duties in regard to sanitation, especially the Quartermaster Corps and the Corps of Engineers. The Quartermaster Corps is normally responsible for such sanitary measures as the operation of water systems, sewage disposal plants, delousing or disinfecting installations, camp or station incinerators and dumps, when two or more units are concerned; and for the actual field work involved in the control of mosquito breeding areas. The responsibility of the Quartermaster Corps is limited to the continental United States, in times of peace, and to the interior areas, in time of war.

The Corps of Engineers assumes most of these obligations in the theater of operations, where it is practicable to do so. When such is not the case, these activities are carried on by the units concerned. In the stress of actual field conditions, water and waste disposal facilities must often be improvised by the local commander.

Water Supply. The first consideration in all water supply problems is the amount of water available. If an adequate amount cannot

be procured locally, it must be obtained elsewhere and brought in by truck, tank cars or pipe lines.

The water supply requirements may be estimated as follows, but the actual amount depends on temperature and other factors: Per man, per day, in combat, minimum for 2 or 3 days, 1/3 to ½ gal.; normal, in combat and minimum in bivouac, 1 gal.; normal in bivouac, 3 to 5 gal. for drinking and cooking; in temporary camps, 5 to 20 gal.; field hospital, 25 gal.; semi-permanent camp, 30 gal.; permanent camp, 50 to 200 gal.; permanent hospital, 200 gal. The amounts required for horses and mules vary from 3 to 5 g.p.d. minimum for a short period to 10 g.p.d. Water consumption per motor vehicle will be ⅛ to 1 g.p.d. per vehicle, depending on terrain and size of vehicle; locomotives, 150 to 200 gal. per train mile. In addition, in fixed camps each shower bath can be considered to use 300 g.p.d., each toilet 40 g.p.d., each lavatory 20 g.p.d. and each urinal 40 g.p.d., but the use depends also on the number of persons using these facilities, on temperature, and on other factors.

When the water supply available is adequate in amount to meet such of the needs as must be provided for, and there are several sources from which the necessary quantity of water may be obtained, selection is made of the water with the most desirable characteristics. Consideration is given to safety or freedom from serious or excessive bacterial contamination; palatability or acceptable taste, odor and temperature; attractiveness, or freedom from excessive color or turbidity; and, in some cases, mineral characteristics affecting convenience of use, as hardness, iron or manganese content and alkalinity. Chlorination of all water is always required, and appropriate treatment must be provided in many cases.

The responsibility for procuring, testing, treating and distributing water is normally divided among the Quartermaster Corps, the Corps of Engineers and the Medical Department. The Quartermaster Corps is responsible for the procurement and treatment of water for all stations and camps in the continental United States, except for any theater of operations. In a theater of operations, in maneuvers and in all field operations, the Corps of Engineers is charged with procuring, treating and distributing water to all major units, down to and including the division, and to smaller units where practicable. Where water is not supplied by the Quartermaster Corps or Corps of En-

gineers, the unit commanders are responsible for procurement and treatment; and unit commanders are always responsible for the protection and control of water that has been delivered to them, and for the equipment used in handling it. The Medical Department recommends sources and methods of treatment and supervises treatment, both in permanent camps and in the field.

Methods of treatment depend upon the situation. In permanent camps, water is filtered, settled and chlorinated, as in municipal practice. For field treatment, portable or mobile filters, pumps, canvas storage tanks, tank carts, water cans, the water sterilizing bag and canteens are provided.

Before the advent of motorized warfare, the scheme of water supply provided for the establishment of mobile purification units, consisting of truck-mounted pressure filters equipped with chlorinators, in the rear areas; the use of tank trucks to haul water forward to distributing points in the rear of front-line divisions; and transfer forward to the troops by means of company water carts. The rapid movement of troops under present conditions, and their greater dispersion, presents new problems. These have been met by the provision of smaller portable units which are designed to accompany the troops into the forward areas, if necessary, and to make greater use of local supplies than was possible with the mobile units.

The mobile unit consists of a truck-mounted pressure filter and chlorinator. The capacity as a filter is 70 g.p.m. and as a pump and chlorinator 100 g.p.m. The portable unit is mounted on skids, and has a smaller filter of the pressure type; capacity is 12 to 15 g.p.m. as a filter, and the unit weighs about 700 lb.

When water is brought forward from the rear, it is distributed to the men from Lyster bags. When local supplies are used, the water is chlorinated in the bag, using ampules containing ½ gram of calcium hypochlorite which, with 70% available chlorine, gives a chlorine dosage of about 2½ p.p.m. In field water supplies, a residual of 1 p.p.m. is required.

Sewage Disposal. In permanent camps and stations, disposal of sewage follows the same methods as in cities, and modern treatment plants are ordinarily installed. In temporary camps, pit-type latrines are preferred unless the soil formation prevents their use, in which case box-and-can or other types may have to be utilized. Where troops

are to be in camp for more than a week, the deep pit latrine is used, soil conditions permitting. Pits are dug 2 ft. wide, 8 ft. long and 3 to 10 ft. deep—the depth being dependent on soil conditions and on the length of time the camp is to be occupied. If the installation is to be used one week, the pit is made 3 ft. deep; and 1 ft. additional depth is provided for each additional week. A linear space of 2 ft. is provided for each seat, and seats are provided for 8% of the command. Therefore, 16 ft. of latrine space is provided per 100 men. Standard latrine boxes, which fit the excavations, are provided by the Quartermaster. Latrines should be fly-proof, carefully maintained and sprayed with oil daily. Usually they are located as far from any mess as possible—preferably 100 yd. away, and about 30 ft. from the end of the company street.

On the march, the straddle trench is used. The standard trench is 1 ft. wide, 2 ft. deep and 8 or 10 ft. long, though trenches may be shorter or longer. Earth removed from the trench is piled at the ends and used for cover.

Urine soakage pits are used in semi-permanent camps where there are no sewers. A pit 4 ft. square and 4 ft. deep, filled with porous material will serve 200 men for an indefinite period. Space at urinals should be provided for 5% of the command.

Solid Wastes. The amount of garbage produced in permanent camps varies from ½ to 4/5 lb. per man per day. Its disposal presents, as in the case of municipalities, a serious problem. In many permanent camps, modern incinerators have been installed of a type similar to those in cities. In temporary camps, field incinerators can be utilized though most of these are wasteful of fuel and require considerable attention. One such incinerator is usually provided for each kitchen. This may be of the cross-trench type consisting of two trenches, each 8 ft. long, 1 ft. wide, and 1 ft. deep, crossing at the center, and with a grate and bottomless galvanized iron can at the crossing. In the field and in camps of less than one week's duration, burial is commonly used to dispose of organic materials, and of tin cans. Combustible material may be burned.

At some permanent posts, as in many cities, garbage is fed to hogs. This almost always results in a serious nuisance in the vicinity of the hog farm and due to the danger to public health from trichinosis and the greater infestation of garbage-fed hogs with this infection,

there is a justified public-health objection to this method. Garbage and refuse dumps are also usually objectionable, but may represent, when properly operated, a satisfactory method of disposal.

Liquid Wastes. The liquid wastes resulting from the preparation and cooking of foods and the washing of dishes and other utensils amount to 1 to 5 g.p.d. per man. In camps of short duration—not more than a few days—these wastes and the solid wastes may be buried. If the camp is to be used for a longer period, soakage pits are required; but if the soil is tight or ground water high, the capacity of such installations is limited, and disposal by evaporation or by other means may be necessary.

Responsibility for Waste Disposal. The Quartermaster Corps is reponsible for the disposal of waste materials and for the construction, operation and maintenance of all permanent facilities and installations utilized for waste disposal by two or more units. Unit commanders are responsible for the proper disposal of waste within the areas occupied by their units and are, therefore, responsible for the construction, operation and care of all temporary or emergency appliances employed by their units for the disposal of waste.

The Medical Department is responsible for sanitary supervision and inspection, and for reporting on and making recommendations pertaining to the construction and operation of all facilities and installations employed for the disposal of those wastes that are directly or indirectly concerned in the transmission of disease.

Insect Control. In the field and in camps of brief duration, measures for the eradication of flies and mosquitoes cannot be effective. Therefore, reliance must be placed on screens, nets and malaria prophylaxis. In permanent camps, such as our present cantonments, drainage, oil and larvicides are utilized for the prevention of mosquito breeding. Plans for the work are prepared by the Medical Department, but the actual performance of the work is a function of the Quartermaster Corps, under the technical supervision of the Medical Department.

Similarly, in fly control, sources of fly production are determined and recommendations for control measures are made by the Medical Department, but the work when ordered by the Commanding Officer is carried out by the Quartermaster Corps. This is the general practice in all insect control work—that the Medical Depart-

ment recommends the necessary procedures for relief, but that the actual work is done by the Quartermaster by order of the Commanding Officer.

Ventilation and Housing. The provision of ample floor and air space is a most important factor in the control of respiratory diseases. In barracks, each man should have 60 sq. ft. of floor space and 720 cu. ft. of air space; in emergency these may be reduced to 50 sq. ft. and 600 cu. ft. Beds or cots should be arranged for head to foot sleeping. However, wide spacing of beds will not compensate for poor ventilation, nor will ample ventilation compensate for inadequate floor or air space.

General Summary. These are but a few of the problems that must be overcome when troops are in the field. As important and complicated as these problems seem, they are less serious and less difficult to solve than some of those that are created by contact between the troops and the civilian population. In the solution of these, efficient local and state health departments are needed, with the fullest and most sympathetic coöperation given and received at all times. The present arrangements contemplate such coöperation, utilizing an officer of the United States Public Health Service as liaison, in order that fullest health protection may be obtained by the coöperative action of all agencies engaged in health work.

EMERGENCY PROTECTION OF PUBLIC WORKS AND UTILITIES†

BY WILLIAM W. BRUSH*

[*Read April 3, 1941.*]

In considering emergency protection of public works and utilities, the question that at first arises is: Against what force or act is protection to be provided? Broadly speaking, the destructive force or act against which protection is desired may be created by man or nature. If the force is man-made, it may be created innocently, due to lack of experience or failure to realize the magnitude of forces acting on a structure, or it may be developed intentionally to destroy or seriously impair the availability of a structure to continue to serve in the manner planned. The forces of nature may be so enormous that nothing that man can devise will effectively overcome them, but what such forces are and how they operate are fairly well known. Also, nature's forces are not malevolently directed against the comfort and security of man. Man-made forces that are malevolent can destroy the functioning of practically all man-made structures, if sufficient time and opportunity be given to those who seek to destroy, due to war or other cause.

Primarily the subject to be considered is emergency protection based upon the destructive action of man, although many of the items discussed would apply in whole or in part to the destructive forces of nature. If we could properly assume that public works and utility plants had been planned and constructed to reduce to the practical minimum the possibility of interruption of service due to forces of nature or forces of man that are not malevolently developed, then we might reasonably consider only those happenings which could arise from acts of war or sabotage. Unfortunately, such an assumption would not be soundly based in the consideration of most of our structures and plants. It is seldom that our designers and constructors have been sufficiently far-sighted to provide for all the happenings that the subsequent operators wish had been covered. Also, financial limitations fre-

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quently prevent coverage of possibilities that have been recognized and considered during the design and construction period. Even if the unusual had been accomplished, and all items had been cared for in accordance with the conditions existing at the time of construction, including those that might be reasonably foreseen, increased demands or other factors that subsequently developed might materially change the effectiveness of the coverage of the structure or system.

Emergency Planning and Organization

While the subject assigned embraces public works and utilities, only water utility systems will be discussed, and each one may make such application to highways, bridges, transportation systems (over or under land or water), public buildings, air fields, docks, communications (including radio, electric, gas and steam generation and transmission), sewage collection and disposal and waste collection and disposal, as his knowledge of such public works and utilities may permit.

In our country, protective planning comes under the nation, state, county and municipality. What may or should be done by the municipality and its agents will be considered in detail with such reference to the larger governmental units as may seem necessary.

An engineer, naturally starts off with a survey. In this case, the survey should be divided into a number of activities that should be brought together to form "A Major Disaster Emergency Plan". This is the name that has been given by the International Association of Fire Chiefs to its emergency plan. Various names and plans have been suggested, and each community should develop a plan that should be backed by as much legal authority as can be given by the municipality acting as a political subdivision of the state. The organization chart for the fire chiefs' emergency plan is shown in Fig. 1. This plan proposes that the Water Supply Committee shall draw its personnel from the employees of the local water department, private water supply, auxiliary water supplies and volunteers. The duties of the Water Supply Committee are summarized as follows: (1) Maintain source of supply; (2) develop emergency supply; (3) maintain supply work; (4) maintain reservoirs; (5) maintain distribution system; (6) develop emergency distribution; (7) control valves; and (8) control breaks in mains.

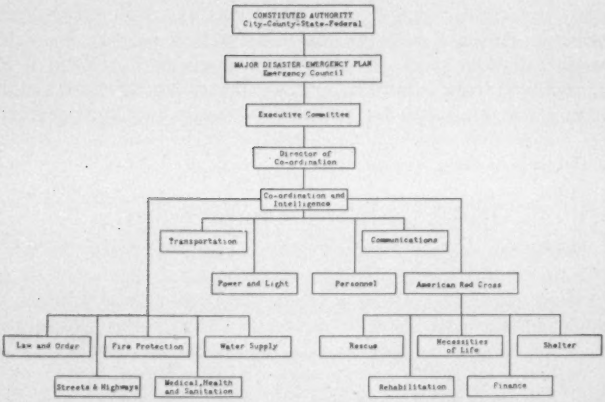

FIG. 1.—A MAJOR DISASTER EMERGENCY PLAN ORGANIZATION. THIS IS THE ORGANIZATION DIAGRAM FOR THE PLAN DEVELOPED BY THE INTERNATIONAL ASSOCIATION OF FIRE CHIEFS.

Under the heading, "Committee on Water Supply", the following is stated:

"Responsibility for the maintenance of water supply, both for domestic and industrial operation, and for fire protection purposes, will be the heavy responsibility of this committee. Under a municipal plan, the direction of the committee should fall to the active head of the normal Water Supply Division, although on a state plan this function may come under a state commissioner. The scope of operation will include not only public water supply, but such private auxiliary water supplies as may be obtained from industrial concerns, private wells, rivers, lakes or emergency storage basins. Consideration must be given to the possibilities of a complete break down of local water supply or distribution, with the resultant necessity for importing water by tank car, truck or emergency pipe line. Provision must also be made for the control of distribution in order that wastage from broken pipe lines and connections may be stopped quickly. In cases where distribution systems may be disrupted, needed water must be transported to the concentration camp sites. Preliminary surveys of the existing distribution and supply systems may develop marked weaknesses, particularly in the event of a calamity. In such cases, special construction should be undertaken, before an emergency is created, to by-pass these weak points. Complete plans for the mobilization of personnel and equipment should be drawn up in advance."

What New York City has done in the way of disaster and war

emergency planning was reported by New York Fire Commissioner, John J. McElligott, at the Annual Conference of the International Association of Fire Chiefs, held at Spokane, Wash., in August, 1940. A Disaster Control Board is set up under the Mayor, as Chairman. The members of the Board are the heads of the various city departments, whose work covers the following subjects: (1) Water supply and power; (2) law enforcement; (3) transportation; (4) communication; (5) health and hospitals; (6) public welfare; (7) fire and emergency; (8) sanitation; and (9) schools.

Three phases are considered, the first being the general disaster situation; the second, pre-war emergency planning; and the third, war emergency situations. For the second and third phases, additional members are to be added to the Disaster Control Board, due to additional organizations entering into the picture.

In the surveys made of the city, under water supply, a study was made of the source, layout and pumping stations of the water supply system. Emergency equipment and personnel were listed. The capacity of auxiliary water resources in different parts of the city, in the nature of pools, cisterns and tanks was estimated and compiled. As a result of this survey, plans were made to replace parts of the water main system with a temporary layout of 4½-in. light rubber-lined hose. This hose was to be held ready for such emergencies on trucks placed at various points throughout the city.

Complete drawings of the water main system were prepared for each Fire Company, and special men, assigned to each division, are to keep these drawings up to date; also, master maps are to be maintained in fire headquarters and in the files of the Advisory Council on Survey and Planning.

Assuming that a general emergency plan has been adopted and the personnel for carrying out that plan organized, with each individual instructed in his or her duties and familiarized with carrying out such duties by suitable drills, we may then consider what the water department organization should do to protect the plant and to be ready to carry out its part of emergency activity.

There can be no fixed order in which the various items should necessarily be undertaken, and presumably, if the size and character of the force permit, a number of the items will be carried out virtually simultaneously.

Maps and Plans To Be Duplicated and Made Readily Available. At least two complete sets of prints of all maps and plans that are considered essential should be made and deposited in such locations as seem likely to be most accessible to those who may need to use such maps and plans in the future. In selecting the maps and plans, the question of their essentiality may be resolved by considering whether they would be required in case it became necessary either to repair or to replace any structure or equipment.

In a small system, it may be desirable to have each set of prints kept together, with one set located at one point, and the other stored at some disance from it. In a large system, it would undoubtedly be advisable to have the prints separated into groups, with the groups located at various points in the area covered by the water supply system, so that it would not be necessary to go over literally hundreds of prints, in order to secure those that would be applicable to the particular part of the system for which the maps or plans are desired.

For the distribution system, a set of prints showing locations of all water mains, valves, hydrants and connections between mains, should be available in every automobile used in the maintenance or operation of a water supply system, although, in very large systems, there would undoubtedly be some automobiles connected with distribution work that it would not be necessary to so equip.

It is also desirable to have fire headquarters provided with maps of the entire distribution system and a skeleton map of the water system. Each fire company should have a map of that part of the system covered by the company, and it should keep such maps up to date. This procedure is the one adopted for New York City.

On distribution and other maps, where reference distances are used to locate structures such as valve boxes, special effort should be made to select reference points that would probably be recognizable, irrespective of the extent of destruction in the area involved.

Checking the Personnel. The most thorough investigation should be that directed toward the employees of the Water Department. The investigation should include every person who regularly or occasionally has access to the water works plant in any of its parts. This investigation should include the ancestral background, the persons with whom the employee associates, including those in his home, and a determination of the political and racial views held by the employee, as indicated

by all data obtainable. Any employee who is suspected of being disloyal, should either be separated from the service or else, so assigned that he could do no damage to the plant. Generally, in municipally owned and operated water systems, only citizens may be employed, but in private water companies, non-citizens may be employed, and special attention should be given to such persons.

Visitors to water works property should be under definite control, whether they are employees of those who are delivering equipment or materials or persons who may come to any part of the plant for other purposes. Unless the purpose for visiting the plant is clearly a reasonable and proper one and the person requesting the authority to visit is sufficiently well-known to justify the issuing of a permit, access should be denied. If passes are used, they should be limited for the day of issue and, at least, bear the signature of the permittee as identification preferably also showing his photograph.

The employees of the plant should be made to realize that they are guardians of the safety of the plant as well as being operators or members of the maintenance force, and they should be required to report immediately any item that is observed, either as to individuals or parts of the plant, that seem to be out of the ordinary. Whether guards should be employed, is a matter about which there will be a decided variety of opinion. During the first World War, a regiment of soldiers guarded the Catskill and portions of the Croton water supply systems of the City of New York, but the regular supervising employees of the city water department were generally of the opinion that such guarding was not effective. In Canada, placing of guards was generally adopted at the outbreak of the present war, but after some six months had passed, many of the guards were dispensed with.

Strengthening the Weak Parts of the System. While the weak points in each water supply system are generally known to the superintendent in charge, at times when emergencies are more likely to occur than under normal conditions, a reconsideration and survey of all parts of the water supply system, becomes an important duty. Wherever there is a single link in the chain that is required to be used to carry on safe operation and such link might be either destroyed or temporarily made unusable, an effort should be made to duplicate the link or in some way make it feasible to continue operation, even though the particular link in question should be seriously damaged or destroyed.

Here financial questions enter into the picture, but the superintendent should present the situation, as he sees it, to those responsible for the finances of the community, as there is not likely to be any single item that is more vital to the community than safeguarding the continuity of a safe water supply.

One item, that is frequently overlooked, is dependence upon electric power for operation of equipment. It may be only a minor part of the equipment that is dependent upon continuity of electric power supply for its operation, but if such minor part fails, it may prevent a major operation being carried out successfully. An entirely independent source of power should be made available, either in the form of a portable power unit or by installation of a fixed power unit with an independent fuel source.

A general item that is today rapidly increasing in importance, is the stock of parts, including materials for replacements. This is due to the increasing difficulty and delay that are likely to be experienced in securing items that may be ordered, and also due to the greater importance of maintaining an uninterrupted and safe water supply during the defense emergency period and during the war, if that should come.

It is not necessary to become panic-stricken and attempt to order everything that would be necessary practically to duplicate the existing water system, but a list of all the items that might reasonably be required should be prepared and such items secured in suitable quantities as rapidly as practicable. Furthermore, it should be kept in mind that the stock should be divided into two or more parts and located at two or more points that are selected both for their accessibility for department employees and their relative safety against destruction by sabotage or bombings.

Where several communities are so situated that an exchange of stock might be readily consummated, it would be economical for such communities to develop a schedule of stock to be carried and to store the stock at such different points as seemed advisable. The stock would then be equally available for each community in case of emergency.

We have not formally entered the war, but we have gone far enough so that we should consider the conditions that would exist if war were to come as well as the conditions that exist due to the presence in our country of many persons who are antagonistic to the American way

of life and who wish to interfere with the preparation for defense of that life.

Protecting Source of Supply. If the source of supply is surface water, the superintendents in New England have much less difficulty in keeping people away from such water than is generally the case throughout the rest of the country. In New England, it is a time-honored practice to endeavor to keep people away from surface waters that are used for public supply. In Massachusetts, the Department of Public Health, through Dr. Paul J. Jakmauh, Commissioner of Public Health, has, during 1940, sent out four notices in reference to allowing access to surface waters used for domestic supply. In a notice issued under date of March 13, 1940, the possibility of pollution of water supplies by the landing of seaplanes on surface waters was set forth, and the amended resolution in reference to boating, fishing, or the landing of seaplanes on a body of water used as the source of public water supply, was given. Under the date of June 13, 1940, Dr. Jakmauh called attention to a recommendation by his department that reads in part as follows:

"... that adequate supervision be maintained to prevent trespassing in the vicinity of pumping stations and reservoirs, wells, springs and filter galleries, or others sources, from which water is taken directly for water supply purposes and that all filters, chlorinating apparatus and other works for purifying or disinfecting the water supplied, be adequately maintained with sufficient extra parts, cylinders of chlorine, etc. for use in case of emergency.

"The Department also recommends that adequate protection be given to power lines where electric power is the only power available for pumping water and that consideration be given to the installation of auxiliary power sources when necessary.

"The Department further recommends that consideration be given to emergency or auxiliary connections with the distribution systems of other adjacent public supplies and in this latter connection the Department is arranging for an engineer to call on you in the near future.

"This Department will be pleased to assist you further in these matters at any time upon request."

Under date of November 27, 1940, Dr. Jakmauh directed "attention to the necessity of maintaining careful supervision over all cross-connections, especially in the form of check valves between polluted water supplies and the water supply system under your direction, and that, in this connection, you notify the industries maintaining such cross-connections of the needs of careful supervision at all times. The

Department again recommends that all improperly protected cross-connections be removed."

Under date of December 23, 1940, a notice was sent out of the action taken by the Department of Public Health at a meeting of the Public Health Council held on December 10, 1940, said action relating to the granting of permits for access to or onto direct sources of water supply, and reading as follows:

"Voted that, acting under the authority of Section 160 of Chapter III of the General Laws, for the purpose of further preventing the pollution and securing the sanitary protection of waters used as sources of water supply, the Department of Public Health hereby revokes any delegation heretofore made by it of the authority to grant and withhold permits for fishing in, or sending, driving, or putting, any bird, fowl or animal into, or for entering or going, in any boat, skiff, raft, seaplane or other contrivance on or upon, or entering or going upon or driving any animal upon, the ice of, or for entering upon for the purpose of cutting ice, or for cutting or taking ice from, any lake, pond, reservoir, aqueduct, stream, ditch, water course or other open water which is taken directly by any city, town, water supply or fire district or public institution for domestic use."

In addition, the Massachusetts Department of Public Health has set up a State Emergency Plan which has been worked up in great detail. This plan has been mimeographed and sets forth the zones and sub-zones into which the state has been divided, shows diagrams of the State Department of Health, Medical Division and Sanitary Engineering Division organizations, and gives a tabulation of personnel, addresses, telephone facilities and equipment of both the Medical and Sanitary Engineering Divisions, and includes private concerns that might furnish various items that may come into consideration in a water supply emergency. The presentation of this plan requires some seventy typewritten mimeographed sheets.

These data have been given to indicate the thoroughness with which the Massachusetts Department of Public Health has endeavored to do its part for the protection of the public health and the comfort of the citizens and is indicative of the care with which these subjects are considered in the New England area. While all permits were cancelled for fishing, cutting ice or entering on water taken directly for public supply, the Department of Public Health subsequently granted permits for cutting ice, but prohibited cutting closer than 250 feet from any intake or permitting any workman to approach closer than this distance.

The permit provided also for constant supervision and twenty-four-hour advance notification of start of work.

The municipal water supply system of New York City is mainly derived from the Catskill and Croton water sheds. Numerous reservoirs are utilized for the storage of the surface water. In the Catskill System, the water from the Ashokan Reservoir, which is the main storage reservoir, flows some seventy-five miles through an aqueduct to the Kensico Reservoir, where approximately two months' supply is stored. When the water leaves the Kensico Reservoir, it flows virtually directly to the city, passing either through or under an open distribution reservoir, holding about one and a half day's supply. In the Croton System, the water finally enters Croton Lake and flows thence directly to the city, with two open distribution reservoirs, connected to the system.

Under the New York State law, the city of New York is required to allow boating, fishing and cutting of ice on the city-owned reservoirs of the Croton and Catskill systems. Access to these reservoirs is under permits that may include such reasonable regulations as the city may impose. Towards the end of September, 1940, Governor H. H. Lehman, at the request of the New York City officials, declared that an emergency existed and hence authorized the closing of the New York reservoirs to the public. Thereupon Mayor LaGuardia cancelled all outstanding permits, and on October 16, 1940, Commissioner of Water Supply, Gas and Electricity Joseph Goodman, issued new rules and regulations under which permits for fishing and boating and cutting ice could be secured. Such permits were limited to residents of the State of New York who were required suitably to identify themselves, to furnish a picture to be attached to the license, and to be fingerprinted. The new rules prohibited coming within 500 to 1,500 ft. of any of the dams and structures connected with the reservoirs, with the exception of the Croton and Kensico reservoirs, where the prohibited areas extended some two to three miles from the location of the dams and intakes. The Governor originally established the emergency period until March 31, 1941, and has, in March, extended it for another year to March 31, 1942.

One item that is emphasized in virtually all discussions and articles on the subject of emergency protection of water supply, is the securing, if possible, of one or more auxiliary supplies. This may mean connecting up a water supply system with that of an adjoining area, utilization

of a private water supply system, or at least providing for such utilization when the private system will meet the sanitary standards required for a public supply or can readily be changed so as to meet these standards. Here a cautionary note should be inserted as a warning against indiscriminate cross-connection of supplies. It is better to have no supply than to have one that is likely to cause illness among the consumers.

Where the source of supply is a reservoir created by the construction of a dam, and especially if the dam be of earth, the breeching of the dam may cause erosion of the dam and its virtual destruction. A supply of sand bags should be found helpful in such an emergency, and the maintenance of the reservoir at the lowest level consistent with safety for the continuity of the supply greatly reduces the likelihood of the destruction of the dam, should a breech be made by a heavy charge of explosive.

The possibility of placing explosives in the interior of a dam at such locations that the blowing out of portions of the dam by explosives would be disastrous, requires special consideration. Where a manhole forms the access to supply structures, such as delivery lines and wells, the elimination of the upper section of the manhole and the covering-over of the manhole with earth, after having obtained measurements from reasonably permanent reference points, is a precaution that can usually be taken. The fencing in of structures and the placing of flood lights, is advocated by many and is an aid to prevent access to the structures by those interested in sabotage. On the other hand, such lights would be a danger in case of war and attempts at bombing water supply systems. It should also be kept in mind that flood-lighting is usually dependent upon the integrity of the electric power lines. If flood-lighting is contemplated, a lighting expert of the power company should be consulted and mistakes thus avoided.

A precautionary step to be taken by employees at supply works, would be to note if airplanes appear in the locality and promptly to report to the proper authority such airplanes and their identifying marks. Suitable means of communication other than by telephone should be established, so that messages can be sent through, should the telephone system be made unavailable. A radio system with a power source other than the local electric service, is to be considered, with messenger service as a possibility if other means fail.

Treatment Works. Treatment works and structures might be considered in the same general category as works connected with the supply of raw water. However, there is the question of disinfection and also of neutralizing any poisons that might be introduced into the supply.

Chlorine is the disinfectant used in a very large percentage of plants, and special attention should be given to all of the equipment connected with the chlorination of the supply, including the quantity of chlorine to be kept on hand. This should be ample to provide for relatively long periods when shipment of chlorine may be interfered with. Priorities on chlorine are likely in the future and would keep stock on hand to close limits. Portable chlorinating equipment should be made available. It should be of such size that it could be regularly employed in case the regular equipment was disabled.

From the disinfection angle, the developments of the last two years in the addition of chlorine to the breakpoint makes it well worthwhile to consider whether one should not adopt breakpoint chlorination and thus destroy not alone bacteria but all other organic matter in the water. By carrying as high a chlorine residual as is feasible in the distribution system, presumably with the aid of ammonia, the organic matter already in the distribution pipes can gradually be oxidized. When this result has been accomplished, the chlorine residuals in the water become available to attack and destroy any bacteria that might be surreptitiously introduced into the water supply, as all of the organic matter that will otherwise combine with the chlorine will have been previously destroyed.

So far, no word has been received from Europe of the use of poisonous liquids, gases or solids or disease-producing bacteria in public water supplies. It seems to be rather generally agreed that the danger from the use of substances such as mustard gas, Lewisite, or gases producing arsenical smokes would not be likely to be serious. It has been suggested that a practical way of determining whether there has been introduced into the water supply any poisonous material that might pass the filters is to let filtered water run continuously through an aquarium, as fish are susceptible to poisons that might be introduced into the water.

Protecting the Distribution System. The distribution system appears to be the portion of a water supply plant that is most likely to be affected by war, especially by air bombings. Whether there would be

any attempt at sabotage is problematical, as the damage to a water system caused by sabotage would generally be noted immediately and corrected.

Reinforcement of weak parts of the system is an activity upon the desirability of which all can agree. This should include the installation of valves, wherever they appear to be required, and it should be kept in mind that destruction of a valve will mean that the water on both sides of the damaged valve must be shut off. It is especially important to have adequate valve installations and sufficient hydrants in the vicinity of industrial plants. If the size and location of all mains is not recorded on suitable maps, including the location of valves and hydrants, this work should be carried out without delay.

Enough men should be trained to maintain and operate the distribution system, including the closing of valves, so that the force available will be sufficient to cope with any emergency that might arise. Under war conditions, it is probable that some of the emergency crews on the distribution system will have to be kept at headquarters and that dormitories and kitchen equipment will have to be provided for their care.

For quick repairs, special devices have been developed and un-

FIG. 2.—WATER MAIN IN BARCELONA, SPAIN, BROKEN BY BOMB. THE CRATER CAUSED BY THE BOMB IS PROBABLY FILLED WITH WATER AND SEWAGE.

doubtedly additional developments will be made, so that repairs can be made without cutting off the jagged ends of broken mains and without cutting pieces of pipe that will exactly fit between the broken sections.

If a bomb explosion breaks a water main, it is likely also to break sewers and house connections, so that the hole will be flooded not alone with water from the main, but also with sewage (Fig. 2). Under such conditions, heavy chlorination of the pipes that have been exposed is necessary (see Fig. 3). Under war conditions, only a relatively short

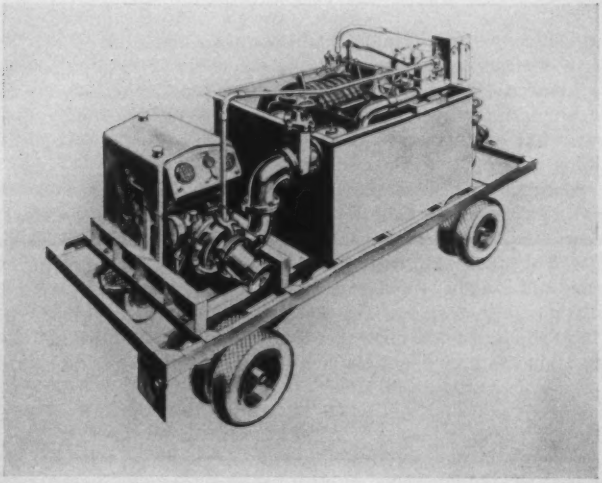

FIG. 3.—EMERGENCY PUMP, FILTER AND CHLORINATION UNIT. THIS MACHINE IS OF ENGLISH DESIGN, TEMPORARILY TO TREAT WATER THAT IS CONSIDERED UNSAFE.

time will be available within which to allow the chlorine to react with the polluting matter. For disinfection under emergency conditions, material of high chlorine content, should be made available in ample quantity to take care of any anticipated requirements.

The cross-connection of adjacent distribution systems has already

been referred to and is something that should be taken care of without too much emphasis being laid upon the question of compensation for the reinforcing of one system by another. Where water supply is cut off in a given area, it becomes necessary to provide water by means of tank wagons or by temporary taps from which people can secure enough water for domestic use. These are matters that require definite planning in advance of need.

Pumping Stations. The protection of pumping stations against sabotage is similar to that of other structural parts of the water supply system. The destruction of the pumping plant by bomb explosions is best met by having a secondary pumping plant located at least one hundred yards away from the main pumping plant. It is also helpful to protect the machinery against the destructive effects of bombs and debris that are likely to fall on the machine, by erecting a heavy timber roof or platform over the various parts of the machinery, so that there will be a minimum likelihood of damage in case of collapse of the building.

An important item is the providing of shelters for employees. These shelters are particularly valuable in maintaining the morale of the personnel and so far as possible, should be made at least splinter-proof. They probably cannot be made proof to a direct hit. Such shelters can usually be constructed in some corner of an existing building, or possibly outside of the building. For the distribution crews, a portable shelter for the crew that is working in an area where bombing is going on has been found to be desirable based upon English experience.

Effects of Bombing

When a bomb explodes, it creates a pressure wave in the atmosphere which may travel at a rate of about 4,000 miles an hour. It is estimated that the pressure wave can persist at any point for about a thousandth of a second and that it is succeeded immediately afterwards by a negative pressure or suction, which appears to last a little longer. Frequently, the suction action will be as destructive as the pressure action.

It has been found that the blast from a 500-lb. bomb will knock down an ordinary house at twenty-five yards distance, while a one-ton bomb will do the same at fifty yards. This is assuming that the

bomb bursts on the surface of the ground, but if it buries itself in the ground, most of the effect of the blast goes upward. Bombs weighing 2½ tons are now coming into use and are estimated to have a destructive zone approximately 3,000 ft. in diameter. It is estimated that a ton bomb, dropped from an elevation of 20,000 ft. would probably penetrate the ground from fifty to sixty feet, and that if the explosion did not take place until penetration was completed, the crater width would be from two to three times the penetration. In full-scale tests of the effects of a 500-lb. bomb, it was found that all water mains were shattered, the joints and steel mains were damaged for considerable distances, and sewers two-brick thick were cracked.

A glimpse will be given of what has happened in England to water supply systems as a result of air bombings, and some of the precautions that the English water purveyors have taken to meet the destructive effects of such bombings.

In a residential town in which an evacuation order had been issued in the summer of 1940, the "alert period" sometimes covered most of the hours of daylight, with bombing and gunfire occurring spasmodically throughout the day, thus retarding outdoor work. In this town, it took a week to relay 72 ft. of 18-in. main "owing to difficulties of water, debris and incessant air raids while repairs were in progress. The last named difficulty may well prove to be a serious one where important mains cross a 'target area'. In this case, the actual relaying only took a few hours and could not have been materially accelerated by the use of mechanical joints or other professional appliances. The situation of the town necessitated a strict black-out, and night work would be so difficult that it has not yet been attempted. Bombs and shells from long-range guns are liable to arrive unannounced during the 'all-clear', depriving the words of much of their former meaning."

The distribution system included three large and three smaller trunk mains and the value of cross-connections and suitably placed valves was shown when the larger trunk mains were put out of action by bombs, and within two to three hours an alternative route was provided.

In England, the men who operate the valves are called, "turncocks". In reference to the turncocks and the repair men the following is stated:

"In this area, only eight turncocks are employed, including the foremen and assistants. Two of them reside at the office, and one or other of these two is always on call except during office hours, when both are out in the district. The turncocks are divided into two watches of four, each watch being on call for a week at a time, during which they remain in the town. When a warning is sounded, they carry on with their ordinary work, unless it is necessary to take cover. If bombs fall in the district, and only then, do they have to report or telephone to the office. This arrangement has been found to work well in practice, and it has the great advantage that the turncocks do not become tired or stale from endless periods of 'standing to' ".

In a community having a population of about one-quarter of a million people, covering an area of some seventeen square miles, a number of mains from three to twenty-one inch had been damaged, but immediately emergency supplies were obtained from adjoining mains. Here the following was stated:

"Experience has shown clearly that the type of temporary pipework with flexible joints which could be laid around a crater would be the most useful and, in many cases, the only practicable solution, owing to debris in the crater and the necessity for repairs to sewers or other services below the water main."

In this community, the water purveyor considered the important items to be the provision of small cars for the turncocks, to have available hydrant standpipes fitted with taps to supply consumers when water mains are put out of service, to decentralize central stocks of pipe and other stores, to have an accurate mapping of the distribution system and to have duplicate copies of all valuable records stored in a "safer district".

Another water purveyor had set up eleven patrol stations, distributed throughout the town area, manned by thirty-five regular employees and thirteen auxiliaries recruited from local master plumbers' associations. The stations are generally near air raid wardens' posts and in direct telephone communication with the water works liaison officer at the central A.R.P. (Air Raid Protection) patrol. The men go to their stations whenever the "alert" warning is received and have full equipment for all emergency work that might be required. The frequent operation and greasing of the valves is stressed as being most important, so that the water may be quickly shut off from broken mains.

In an address, a Sanitary Inspector in Scotland emphasized the following:

"1. The frantic preparation of proper and up-to-date plans of local water systems was in only too large a number of cases one of the first effects of the war.

"2. The Department of Health called representatives of water undertakings together to devise a scheme for mutual assistance. As a result, materials and special plant were pooled and deposited at convenient depots, thus making a considerable saving for the ratepayers. Alternative means of supply were investigated and, by means of cross-connections, many adjoining undertakings obtained an alternative supply.

"3. Squads of men were furnished with equipment and tools for emergency repairs, and were made available for a number of nearby water districts.

"4. Special attention was given to sterilizing broken mains and the new lengths of pipe used in making repairs.

"5. An effort is made to see what water mains are up to normal capacity by cleaning, and if necessary, re-lining the mains that have become tuberculated.

"6. Water waste prevention is stressed and frequent house-to-house inspection advocated. In this connection, it was stated, 'Many householders acting on the advice of the Ministry of Home Security to keep the bath full for fire fighting purposes fill the bath each night and empty it again in the morning. This erroneous interpretation has resulted in a very serious waste of water. What was intended was that used bath water should be left in the bath until the bath is next required.' "

The quotations given are from the February, 1941 issue of *Water and Water Engineering*, London, England.

A picture of what is happening in the sterilization of repaired water mains in the bombed areas in England, is given in an article by Lt. Colonel E. F. W. Mackenzie, Director of Water Examination, Metropolitan Water Board, London, England. This article appeared in the December, 1940 issue of *Water and Water Engineering*.

After noting that sterilization of repaired water mains is a practice that has not been generally followed in England to anything like the extent that it is in the United States, Col. Mackenzie states that experience has shown that a high-explosive bomb bursts a water main, may blow considerable debris into the main, at times sufficient to completely plug up the main. Also, that house sewer connections are usually broken or caused to leak sufficiently to present the problem of sewage being present in the water that fills the bomb crater.

To sterilize the main, mobile chlorinators are used, the one designed for London providing accommodations for six chlorine cylinders, mounted on a two-wheel trailer, with a gasoline-driven engine to provide the necessary pressure for injection where a water supply under pressure is not available. It has been found from experience, that one

automobile is required to serve two of these trailers. The trailer is taken to the point where sterilization is to be carried out and then the automobile usually goes to serve a second trailer.

Originally, an effort was made to sterilize with a solution of chlorine with a concentration of only 1 p.p.m. and an hour's contact, but very quickly, it was decided to increase this dosage to 10 p.p.m. irrespective of whether sewage did or did not appear to be present. In London, a chlorination supervisor is assigned to each district and has two of the mobile chlorinators and one chlorine vehicle at his disposal. Each of the mobile chlorinators is staffed by a chlorination attendant. The minimum time of contact of the chlorinated water with the main, after the main has been fully charged with the chlorinated water, is set at fifteen minutes. As far as practicable, samples have been taken from mains thus treated and have shown no *B.coli* in 100-ml. samples.

Where contaminated water has been in contact with a long stretch of main, the chlorinated water is allowed to stay in the main for a period that may be extended to several hours. In a 16-in. main, where the bomb fell at the crest of the hill and both the main and the sewer were broken, the main extended downhill on one side for one-quarter of a mile, and on the other side for three-quarters of a mile. The mixture of sewage and water filled this main, and when it was repaired and the water was drawn out of the main, the main was filled with chlorinated water which was allowed to stand for several hours in the main. After the main went back into service, tests showed no *B.coli* in 100 ml, while the twenty-four hour, colony count at 37°C. was 2 per ml.

Summary

The experience in Europe and the consideration of the problem of protecting utilities in this country indicate that probably the most important item is to have a personnel as to whose loyalty there can be no reasonable question. Next in importance is to bring the water supply or other utility system up to as high a standard as practicable in the way of providing against serious interruption of service, due to damage to any part of the system. Then comes the stocking of an ample supply of materials and equipment with which parts damaged can be expeditiously repaired or replaced, and in the case of water supplies, utilization of other sources of supply that may be possibly made avail-

able and that would be safe sources for use. Beyond these items, there are less important ones, but it is believed that finally, reliance must be placed mainly upon the ingenuity and fidelity of the personnel after the precautionary measures have been taken that seem to be applicable to the particular plant that is under construction.

COOPERATION OF FIRE AND WATER DEPARTMENTS IN PLANNING FOR EMERGENCIES

BY SAMUEL J. POPE*

[*Read February 20, 1940*]

There are no other departments in a community that are more closely allied than the water and fire departments. The water works is the right arm of the fire service. In the city of Boston we have a very good set up. Daniel Sullivan, Division Engineer of the Water Department, has always worked in close coöperation with the Fire Department. I know that many problems confront water works engineers besides furnishing a sufficient water supply at adequate pressures for fire service. However, it is this service with which the fire chief is mostly concerned. I believe that when new water lines are contemplated, it would be of great advantage if the water works engineer would call the fire chief for a conference and make known his plans. At this conference many errors could be avoided. A fire chief who is on the job will be able to anticipate how much water is needed, at what pressure and what are the hydrants necessary for a given location, having in mind always the factor of safety in the event of a change in the district. He will also know the number of pumpers and lines that could be efficiently operated from these water mains. I believe that it is of the utmost importance that every fire chief should know exactly what he has to work with underground. In Boston, maps showing the water system are always furnished the chief officers. Frequent use of these maps is made by the officers when instructing the men. However, the practical experience of sitting down and talking matters over with the water works engineer, particularly with reference to new work, should prove invaluable to both departments. Many times, work or extension of water lines is contemplated in certain sections when, as a matter of fact, there are other sections of the city or town that, from a fire viewpoint, should receive priority. Here again a conference would be of lasting benefit. The proper staggering of hydrants is a most important factor for the fire de-

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partment, particularly in these days of heavy traffic on thoroughfares.

In these times of emergency, when we are all so concerned with the proper defense of our nation, the water department and fire department become very important. One glance at what has happened on the other side of the ocean gives us much food for thought. It is imperative that proper water supplies and auxiliary supplies be anticipated and provided. Should damage occur to some of our water mains, we know that by the proper closing of gates, the damaged section could be isolated and water still supplied through the water system. However, I do not believe that the fire departments of this country are suffciently informed of the location and proper operation of these gates in time of emergency. I am of the opinion that certain emergency crews of the fire department should take special instructions from the water works engineers concerning these gates and that they should be provided with the proper equipment in order that they may be able to act in an emergency—instead of waiting for the emergency water department crew, which may be engaged elsewhere—and thus avoid the loss of valuable time. Every chief in the fire department should know the exact location of gates just as well as he knows the location of hydrants. In addition, gates should be legibly marked, and every member of the fire department should know and be familiar with these markings. It is my opinion that the marking of gates should be uniform throughout the country. There are many instances when a department is called to render assistance to another community. Thus, uniform marking would prove to be invaluable just as uniformity has proved to be such a success in fire department threads.

There are many private fire-fighting systems that take their water supply from the public distribution system. By a private system, I do not mean the ordinary standpipe or sprinkler system that is found in single buildings. I refer to mains and hydrants that are carried into navy yards, arsenals, railroad yards, large industrial plants, etc. Many of these lines were laid years ago, and in some instances the information available about them is not as complete as for the public water supply. If at all possible, all information concerning such systems should be furnished the local fire chief. At least, flow tests should be made and the results communicated to the fire chief. Here again the location and proper marking of gate valves in such systems would prove of great benefit, because in the event of serious damage

to such properties, it is not beyond the imagination to see how the public water supply might be greatly reduced, both in pressure and volume by open piping.

In Boston, as in many seacoast towns, we are benefited by being surrounded by water, and we have sufficient pumpers to properly utilize that water. Every available water supply of 5000 gal. or over should be charted and be considered as an auxiliary water supply for a fire department. In Boston, in connection with our defense program, we are doing that very thing. Many of our western cities that have had the experience of earthquakes have prepared for such emergencies by providing fire department connections on the outside of buildings, running from private water supplies. As an example, there are many swimming pools and tanks, the water from which could be made available as an auxiliary water supply for fire department use.

It is interesting to note that no matter how much we think we may have advanced over our forefathers, there are times when we are brought to realize that they knew a great deal. Many of the up-to-date practices were used to advantage years ago. Take for instance the old water cisterns. Before the San Francisco earthquake and fire, Chief Sullivan, who was the Chief of the San Francisco Fire Department at that time, was trying to get the city authorities to install water cisterns for emergency service in case of earthquake. He was called a reactionary, and it took that great fire and earthquake, in which he was killed, to prove to the city fathers the value of such emergency service. Today San Francisco is studded with these emergency services.

I do not believe that it is good policy to do away with something just because we have something more modern. When we buy a new pumping engine, we do not discard the old one. We place it in reserve, because we never know when its services may be utilized, particularly in an emergency. The old pump, when new, was capable of delivering 750 g.p.m., which would be three good fire streams. After years of service, it would probably only deliver 600 g.p.m., but we still would have two excellent fire streams. So it is with the old cisterns. If you have them, do not do away with them. Keep them in reserve. They may prove of great value in an emergency.

In our climate there is a likelihood of running into frozen hydrants. When a fire company locates at a frozen hydrant, there

is a grave possibility of very serious consequences. We know that the most serious fires take place because of delayed alarm. If it receives an alarm in reasonably quick order, the average fire department is able to handle the fire. There is one exception to this and that is running into frozen hydrants. Usually the frozen hydrant is due to improper drainage. An excellent practice, which is carried out in the city of Boston, could be adopted also by other communities with good results. When the fire department responds to an alarm and uses a hydrant, a man from the water department later visits that hydrant to make an inspection and see that the hydrant has drained properly. If the barrel is filled with water, he has the necessary equipment to pump out the barrel and thus to eliminate the possibility of freezing. Every pumping engine in the city of Boston is equipped with a Ross thawing device for thawing frozen hydrants, but this takes time. We have a rule in the fire department that if an operator finds a frozen hydrant he is not to leave that hydrant and move to another but is to remain and thaw the frozen hydrant. The reason for this rule is obvious, the second hydrant might also be frozen, and serious delay would result. However, in the winter months, each pumper responding, so far as is practicable, will take separate hydrants. I believe the indiscriminate use of hydrants by contractors should be avoided, particularly in the winter months.

As to the hydrants themselves, Boston has post hydrants, some with independent gates and some without. I understand that the water department has had difficulty with the independent valves and is now installing the ordinary post hydrants without independent gate valves. So far as the fire chief is concerned, this does not offer any problem. When an operator connects a pumper to a hydrant with three outlets, he immediately attaches two Morse gates to the remaining outlets, and additional pumpers may then connect without shutting down the hydrant. We find that the orange painting of the hydrant top affords ideal visibility. It is my opinion that it would be of great advantage to a fire department if some uniform marking of the size of the water main could be placed on the barrel of every hydrant installed.

I have touched on only the most obvious problems that are shared in common by the fire chief and water works engineer. Needless to say, water works men will take steps to protect our pumping stations

and water supplies from any enemy within or outside our communities. I feel quite sure that water works men will acquaint their fire chiefs with all the auxiliary supplies available and guide them in their use and adaptability. As the fire chief of a large city and a member of the various chiefs' organizations in this country, I know that every fire chief will welcome advice and guidance of water works men in this technical problem.

LENGTH OF SERVICE OF WATER WORKS FACILITIES—KNOWLEDGE OR GUESSWORK?

BY REEVES NEWSOM*

[*Read April 24, 1941*]

From one point of view, it is somewhat surprising that, in any sphere of activity that is as fundamental and as essential to life and civilization as the furnishing of water, we are so hazy about how long the facilities used can be expected to continue in service.

We are all familiar with instances where expert testimony varies widely in regard to the life expectancy of various elements of water plants. Not long ago, a Supreme Court referee, in a report rendered in a tax case involving the Croton dam and surrounding lands, commented on the very large differences of opinion contained in the testimony concerning the probable life and depreciation of the dam. One side contended that it would be used for 500 years while the other side maintained that, though built only in 1906, it had already depreciated 34%.

On the other hand, it is perhaps only to be expected that, where such a large proportion of the value of a water plant is in facilities that will last for several generations, the operator, with all his problems of quantity and quality, pressure and dependability, and accounting and collecting, has not spent much time in making records of the performance of plant elements or of analyzing records if they have been kept. The question of how many years such long-lived materials and structures would last was not one of his pressing problems.

Water works operators and engineers, therefore, have, all too often, based their ideas of service life of the various parts of water systems upon relatively short personal experience, largely limited to local and special conditions, without taking the time and trouble to ascertain whether that experience is typical and whether it is safe to generalize from it. As a result, great divergencies of opinion have been expressed on many occasions, not only in regard to the lives of dams, as already mentioned, but also of pipe lines, hydrants, services,

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meters, elevated tanks and standpipes and various other items of special equipment. For a number of reasons, the results of this situation, at least in the municipal field, have not been particularly serious in the past.

Because there has been such a wide margin between the maturity of water bonds and the life of the plant they cover, as compared, for instance, with the highway bond situation, there has been no need on that score to worry about a more exact determination of the probable life of water works facilities. Generally speaking, municipal water works do not attempt to provide for the costs, as such, of replacing plant that is consumed in service. The common practice is to issue bonds for improvements and extensions and then to pay those bonds off from part of the revenue collected for water service. This reestablishes the borrowing power of the department, so that, when replacements are needed, there is ample credit to borrow new funds to pay for them. Thus, without conscious planning, depreciation is being cared for by the rates paid by the consumers although there is no particular relation between the provision for depreciation and the rates.

If the amounts collected are ample to keep up this recurring cycle and perhaps to provide some margin to be turned into the city's general treasury, everyone is pretty well satisfied. At least they are satisfied until the need for a political issue attracts some campaigner's attention to the water rates; then a great many people find fault and the water superintendent is often put on a spot from which he cannot defend the rates with very sound arguments. In such situations, the value of the plant and property is not a factor, the amount of depreciation is not considered and, therefore, in the past, records of what had been happening in the way of service rendered by various facilities has not been a matter of concern in rate determination in municipally owned systems.

While it is not yet apparent in the experience of many water works men, there is unmistakable evidence in many quarters that changes are in store for the water supply industry in matters of this kind. As a result, municipal water departments are going to be faced with certain problems that, heretofore, have concerned only private water companies. The methods of operation that have been followed in the past, even in as relatively stable an industry as the water supply business, are, undoubtedly, becoming outmoded by the changes that are taking

place in the world today. It is to be anticipated that demands will have to be met for justification of many of the things that, in the past, have been done as a matter of course. The raising of funds for water extensions is already in a different category, psychologically, than before the days of P.W.A. The scope of activity of regulatory bodies of all sorts has been vastly enlarged. Rates and the allocation of costs between domestic and fire service can be expected to be questioned even in municipal plants in the era of high taxes in prospect. Almost certainly, some degree of change in the management of water works may be called for, and conceivably it can go beyond anything we now anticipate.

During the last few years, there has been a spread in the movement to place municipal water plants under the jurisdiction of state public service commissions, particularly with regard to rates and the issuing of securities. Due to the general extension of central control by government and state agencies, it is logical to expect that the water works business, even though largely municipally owned, will, in an increasing number of states, find itself a regulated industry. One of the early types of regulation to be expected is that of rates to be charged to customers outside the city limits or to other municipalities. It is becoming common practice for an increased charge to be made outside the corporate limits of the municipality owning the water system. The last revision of the Public Service Law of Pennsylvania provides for the regulation of municipal plants in that respect and, undoubtedly, represents a first step toward the more complete regulation now in force in a number of states. Each year, bills are introduced into several state legislatures putting all water works systems, municipal and private, under the state public utility commission. Some of these bills will pass from time to time, giving impetus to the movement in other states. When regulation arrives, the questions of what the plant consists of, what it is worth, and at which rate it is wearing out are to be met.

There is also an increase in the practice of selling water revenue bonds, entirely divorced from the city's general credit, to finance improvements. With the increased problems of raising money, with which this country and all its political subdivisions will be faced in the future, due to defense requirements on top of heavy present indebtedness, it is reasonable to expect that, in more states, laws will be passed

to enable water departments to finance their needs without affecting, even indirectly, the borrowing capacity of the city involved. When revenue bonds are to be sold against the water plant and its earnings, the question of the equity behind such bonds is of moment; therefore, the value of the plant must be determined, which, in turn, involves the questions of depreciation and service life.

It is going to be necessary for many communities to find new sources of revenue or additional taxable property to meet the rising costs of government and defense and their ramifications. Many cities have reservoirs, dams, large pipe lines and other important works which are situated in outside communities and upon which taxes must be paid; or, if not paid in the past, such taxes may be legalized in the near future. More scientific and consistent methods of property assessment for tax purposes are developing and, as in the case mentioned earlier in this paper, water works systems are going to need factual evidence to resolve fairly the claims that will be made regarding the value of their taxable property.

The problem confronting the water supply industry is the finding of a substitute basis for the more or less glorified guessing that has been done in the past as to how long water works facilities will last. It is no less a problem for municipally owned systems than for water companies. The National Association of Railroad and Utilities Commissioners has in preparation a comprehensive report, or manual, on depreciation principles and practices, as a guide for its individual members. That association comprises practically all of the state utility regulatory bodies in the United States. The other utilities—railroad, telephone, electricity and gas—have, for some years, been co-operating with the committee studying the problem by collecting data relating to the survival and retirement experience of the items of property in their respective fields.

Speaking generally, the water works industry has not awakened to the responsibility of providing factual data relating to the survival and retirement experience of its particular classes of property. In the absence of such data regarding water works the National Association of Railroad and Utilities Commissioners will be predominantly influenced in their determinations by the data regarding the relatively short-lived facilities of the other and more numerous utilities. Determinations so arrived at will almost certainly be detrimental to water works utilities,

including both municipal and private plants, irrespective of whether or not the various commissions now have jurisdiction thereover.

It is obvious that, without regard to how any one wishes to theorize, or what the field of specific use is, the first job in the procedure of finding a better basis than more or less arbitrary assumptions of service life is to ascertain the facts regarding past retirements. Unfortunately, a simple application of the experiences of the past will not determine what the useful life of water works facilities will be in the future. We must recognize the practical limitations in the application of past performances to estimates for the future.

As experience has accumulated, materials and equipment have been better made than in the early days. Pumps of various types have attained such high efficiencies that further improvement can come but slowly, so that replacements for the reduction of operating expenses will be less frequent in the future. Meters and other mechanical devices are also nearer the highest practicable efficiencies. Metal mixtures are more resistant to destruction than has been the case in the past and, generally speaking, methods of preventing corrosion and other forms of deterioration have been invented and put in practice. Wherever waters are particularly corrosive, water mains now being installed are lined and those to be installed in the future will be. Mains already installed can be cleaned, and distinct progress has been made in controlling the recurrence of tuberculation by the treatment of active waters. At the present time, methods of lining pipe in place are available and the success of the limited number of such applications that have been made indicates their more common use in the future. Replacements of pipe, therefore, that have been brought about by the action of highly corrosive waters are not likely to be repeated so often in the future.

In many cities, the records of past retirements reflect the standardization on some type or make of hydrant, valve, service pipe or meter. The result of such standardizing was the replacement of many units that, had they been of the newly adopted kind, could have been economically repaired. In the future, the units in service will not be subjected to this type of retirement to the extent that they have in the past. Furthermore, the slowing down in growth of the total population and, more particularly, the marked cessation of rapid growth of our cities, as shown by the 1940 census, clearly indicate a dropping off

in the anticipated number of replacements of water mains because of rapidly growing territories and, in turn, in the future needs for additional supply sources, filter plants, pumping stations and storage capacity.

But, irrespective of the difficulties in the application of past records to future prognostications, fundamentally, the necessity for ascertaining them remains. Without such data, no intelligent basis is at hand from which such future variations as are apparent can be considered.

When attempting to gather information in the field concerning the installation and retirement of water works facilities, the situation with regard to the condition of such records generally falls into one of three classes:

1. A system has no such records back of some relatively recent date.
2. A system has good records of the installation dates of the facilities now in place but has no records of what has been retired, except in recent years and of the equipment replaced by that now in service. Though the size and type may be known, no dates of installation are available.
3. A system has good records of both installations and retirements from the beginning or near the beginning of the plant's operation, either on cards or contained in a series of yearly reports.

Records of some kinds of property units will be found in certain plants and other kinds in other systems. Only a few have complete records of all kinds. Unfortunately most water systems fall into one or the other of the first two classes.

At some time, perhaps fifteen, twenty, or thirty years ago, the superintendent realized the value of records of performance of various kinds of materials and equipment to guide him in his selection of new facilities when required. A complete inventory of what the plant consisted of was made, together with a compiling of such information as to dates of installation of the plant elements then in use as was readily available. Since the date of the inventory, records of retirements and subsequent additions to plant have been faithfully made; but no effort has been made to find out what happened in the early days of the plant's history, particularly with regard to facilities that had come and gone as a part of the system. For purposes of operation, such old occurrences were not of interest, and, in many instances, old records, from which retirement data could have been unearthed, were destroyed as valueless. As a result, many water works have good records of what

their plant consists of and perhaps the dates when its various present elements were installed, but they possess little or no information regarding things that were once a part of the plant but are no longer in use. Therefore, no complete story of all that has comprised the system and all that has happened as regards periods of service can be put together.

The reason why it is essential to have the records of what has happened to all the facilities that have been in use up to and including the present is that, if we hope to determine the probable period during which a plant's property units can be expected to continue giving service, it is necessary to know not only the age of the facilities that have been taken out, but also the proportion of such retirement to the amount of property that has survived during various periods of time. Otherwise, we do not know what weight to give to the average lives of units of property that have already gone out of service.

The difference between the results obtained by considering only the age of facilities retired and those which consider also the amount and ages of the units that survive is well illustrated by the studies of retirements and survivals of cast iron mains at Springfield, Mass., made by E. H. Aldrich and the writer and reported upon in the *Journal of the American Water Works Association*, and those made by Harry U. Fuller regarding water mains at Portland, Me., and reported recently in the journal of the Maine Water Utilities Association. These two studies develop the method of translating records of retirements and survival of property in service into tables and curves that indicate what could be expected to happen in the future if the factors causing replacements were no less severe than in the past.

In addition to transmission and distribution mains, certain other facilities are present in a water plant in sufficient numbers of units to lend themselves to similar mathematical treatment. Included among such facilities are valves, services, hydrants and meters. The main pipe studies mentioned before are typical of what can be done with data regarding those items of property for which records of installation and retirement can be found covering relatively long periods of time. However, the use of partial data to develop percentages of retirements for short periods and the average ages of property retired during such periods can result in some very misleading conclusions if such percentages and average ages are used as a basis for estimating the life expectancies of the property remaining in service.

At Springfield, for instance, in the case of the 6-in. pipe, the average age of the mains that have been retired is 22½ years. But only 8.6% of the amount installed has been taken out. When consideration is given to the service record of all the pipe that has been put in as well as that which has been taken out during a 73-year period, it is found that the survival rate is over 89% at the end of that time. To say that the 22½-year average age of the small amount of pipe retired is any criterion of the average service life of pipe of that size is an error apparent to anyone. With 8-in. pipe, the average age of the pipe that was retired is 19.9 years. But again only a small part, 3.1% of the total installed, has been retired. When all pipe of that size is considered, the corresponding survival rate at 73 years is just over 90%. For sizes 10 in. and larger, the average age of the 3.5% of the pipe retired is 26 years, but the survival rate after 73 years is 93%.

These figures illustrate also the distinct difference between the ratios of the amount of pipe retired to the amount installed and the survival rates of the pipe that has been installed. Furthermore, this ratio of the total pipe retired to the amount of pipe installed during any number of years of existence of the plant is of no particular significance, because some of the mains recently retired may have been installed in the year in which the plant was constructed while others may have been installed only two or three years before removal and no distinction is made between such retirements in considering the mere total. Likewise most of the pipe of a given size in a system at a given date may have been put in as a part of the original plant and very little since, or conversely, a small amount of that size may have been used at the beginning and the bulk put in only during the last five or ten years. The same conditions may hold for valves, hydrants, services or meters. It can be seen, therefore, that merely dividing the total retired by the total installed gives a meaningless ratio.

Again, in considering this question of survival, the relationship of the number of units retired in any particular calendar year to the number in service at that time is of no moment. In arriving at the survival rate, there is balanced against the total number of units of like property that have remained in service for various periods of time the number that are retired at ages corresponding to those periods of time, without regard to the particular calendar year during which the units were installed or taken out.

Thus, in computing a survival curve, the point corresponding to the end of the first year is determined by relating the total number of units of one kind that have been installed to all such units as were retired within the first year after installation, irrespective of which calendar years were involved. The second point, correspondingly, represents the relation between the units remaining in service after all one-year-old retirements are deducted and the number of units that were retired two years after installation. By continuing this process throughout the period of years for which records are available, a curve is developed, each point on which represents the percentage of a group of property units of the size and kind under consideration that is likely to survive the number of years corresponding to the point selected.

If representative figures of the years of service that various facilities have given are to be arrived at, not only must records be of the character outlined and for long periods, but records from a large number of plants should be made available, particularly with reference to units of which there are but a few in any one system. Units of this latter type include impounding reservoirs, tunnels and aqueducts, wells, purification plants, pumping equipment, tanks, standpipes and distribution reservoirs. These are works which are influenced as to type, size and length of useful life by the particular conditions under which they operate. With certain of these units, the combination of records of different installations would be meaningless, and only ranges of life span can be determined. But even if combining the records of some types of individual units is attempted in order to develop average figures, it can be done only in the light of the variable conditions surrounding their use. Therefore, records from a great many systems are needed in order to have any substantial number of units in similar categories.

In this connection, the writer would like to quote from the discussion by A. V. Ruggles of the New York Public Service Commission of a recent paper by Chas. W. Sherman before this Association on "The Useful Life of Iron and Steel Standpipes," and from the reply of the author thereto. Using the same basic data regarding 111 standpipes, Mr. Sherman found, after making certain assumptions as to remaining life, based upon his observation and experience, an average life of 45 years for the group. Mr. Ruggles, by the use of statistical computations, arrived at 55 years as the average age. The latter had this to say about the value of results based upon a limited number of cases.

"Statistical methods based on the mathematics of probability produce more valuable results when applied to records of placements and retirements over a long period of time and of large numbers of like units, much larger numbers than in this standpipe record. Any result thus obtained from this record cannot be considered as valuable as one obtained from a complete history, if such could be compiled, of say a thousand tanks, including recent as well as earlier installations."

Mr. Sherman made the following comment:

"I am inclined to be skeptical as to the value of the results of statistical computations from fragmentary data. There can be no doubt that, when the number of records is large and all of them are taken into consideration, results of great value can be obtained."

A committee has been formed by the American Water Works Association to attempt the compilation of survival and retirement experience with water works facilities from whatever sources records may be made available. The New England Water Works Association has been invited to join in the sponsorship of the undertaking and to place members on a joint committee. About thirty cities have already agreed to make their records available. These cities are scattered throughout the entire country and include two in Canada. They have a population in excess of 13,000,000 and approximately 23,000 miles of mains, 2,700,000 services, 1,600,000 meters and 160,000 hydrants. Undoubtedly, records sufficiently adequate for the purpose will be found for but a fraction of all these facilities, though with regard to mains and hydrants that fraction should be a material one.

In the New England states, where many water systems are of substantial age and where the art of furnishing water service is highly perfected, there are numbers of cities that have very valuable records. It is urged that, wherever possible, they should be made available in connection with this effort to ascertain facts upon which the industry can stand in question involving the life in service of its plant elements and facilities. Benefits will, the writer believes, accrue in the future to all concerned in many ways not suspected at present, in addition to those which always follow when knowledge supplants guesswork.

THE SAUGATUCK DEVELOPMENT OF THE BRIDGEPORT HYDRAULIC COMPANY

BY SAMUEL P. SENIOR*

[*Read April 24, 1941*]

The Saugatuck Dam is about twelve miles northwest from the city of Bridgeport on a river with thirty-three square miles of watershed. The Bridgeport Hydraulic Company does all its own construction work and is building across the Saugatuck River a dam 120 ft. high and about 1,100 ft. long, with a spillway elevation 280 ft. above mean high water, with a gravity section made of concrete. The foundation is Waterbury gneiss, and a short ways below, where it outcrops on the surface, it is sound and free from large seams and cracks. Bore holes put down to a depth of 60 ft. into the rock have proved this.

The Company last year finished building about 7 miles of highways with bridges, culverts and standard highway-department wire-rope railings. Wood and timber has been cut from the basin and the stumps pulled out for a vertical distance of 20 ft. below the flow line. The Company has completed a concrete dike with a maximum height of 40 ft., and length of 800 ft., along a low spot on the easterly rim of the reservoir to prevent overflow in that direction.

The main dam is now 60% complete, and it is hoped that it will be finished during this calendar year. Gravel for the aggregate is secured about three miles up the valley where the Company has a modern crushing, screening and washing plant, which separates the gravel into three sizes, and separates the sand, all of which are thoroughly washed. Forty per cent of the aggregate in each batch is 2-in. to 6-in. gravel, 35% is ¾-in. to 2-in. gravel, and 25% is ¼-in. to ¾-in. gravel. The concrete is mixed very dry with a ½-in. slump. In the interior of the dam 0.93 of a barrel of cement is used and near the faces 1.07. The concrete is transported from the mixer out over the dam by a Lidgerwood cableway of 15-ton capacity having a movable tail tower, by means of which concrete can be placed in any part of the whole structure.

*President, the Bridgeport Hydraulic Company, Bridgeport, Conn.

Water from this reservoir, which will contain about 12 billion gallons, is to be brought easterly into the Company's Hemlocks Reservoir. Two ridges intervene between the dam site and the Hemlocks Reservoir. These ridges have been tunneled. Tunnels 8 ft. 6 in. in diameter have been driven through these ridges. One of the tunnels is 1200 ft. long and the other 5900 ft. They will be lined with concrete and their diameter will be 6 ft. 4 in. inside the smooth concrete lining.

THE WATER SUPPLY OF LUNENBURG, MASS., OR CAN MY TOWN AFFORD A PUBLIC WATER SYSTEM?

BY HOWARD E. BAILEY*

[*Read November 21, 1940*]

You may read, in the Annual Report of the Massachusetts Department of Public Health, the following statement:

"There still continue to be 96 Municipalities in the State not having public water supply systems. According to the Census of 1935, two of these municipalities have a population of 5,000 or over, five have a population of 4,000 or over, eleven have a population of 2,000 or over, thirty-four have a population of 1,000 or over and 62 have a population less than 1,000."

This may lead you to inquire what reasons may have kept your town from installing a public water system when 255 other municipalities in the state have found the "water business" a profitable one. Is your town so far different from these others that it can never hope to have running water in each dwelling? If world conditions are so uncertain now as to make such a venture foolhardy what was your town doing about a public water supply in 1925 to 1929, when we were "sitting on top of the world?" These and many other similar questions can best be answered by examining the records of what other similar towns have done and are doing today toward securing a public water supply.

The town of Lunenburg, with a population of about 2200, like many another Massachusetts village, is primarily residential in character, a few farms of moderate size, a dairy or two, some apple orchards, but no definite industrial or commercial development. With two cities located within a short distance, Lunenburg might easily have become a suburban residential area for both Fitchburg and Leominster. Development in this direction was retarded probably as much by the lack of a public water supply as by any other cause.

Early attempts at securing a water supply date back to 1915 when agitation for the formation of a private water company was started. Test wells were driven and a potential supply located about

*Consulting Sanitary Engineer, Boston, Mass.

a mile from the center of town, but financial arrangements were not completed and the proposal died. Again in 1929 a study in considerable detail was made for a town system but did not receive the necessary support to develop further. The water works plan was revived in 1938, but so little interest was shown at the town meeting that further attempts to secure a water system for the town seemed hopeless. A group of public spirited citizens however, realizing the needs and, at the same time, the limitations of the town secured by private subscription the necessary funds for an engineering study looking toward the formation of a water district.

A petition for the formation of the Lunenburg Water District was introduced into the Massachusetts Legislature in December 1938 and passed in February 1939, being signed by the Governor on February 14. The bill, which was accepted by the voters of the District at a special meeting held on February 23, contained the usual provisions regarding the formation of such Districts and allowed the District to borrow up to $160,000 by issuing bonds or notes payable in not more than 30 years from their date of issue.

During the period when the legislation was pending, an application was prepared for the first Federal Works Progress Administration project amounting to a total estimated cost of construction of $373,810 of which the District was to pay $95,801.90. Approval of the project was received on May 1, 1939, and work was started on the 15th of the same month. The work was prosecuted rapidly, as many as 435 workers being employed on the project at one time. Briefly, the project included about 12 miles of pipe from 6 to 12 in. in diameter, a standpipe of 250,000-gal. capacity and a pumping station.

Some consideration was given to the possibility of obtaining a ground water supply from wells located at the site of the 1915 explorations, but the long section of force main required and other considerations made a proposal for taking water from the City of Leominster much more attractive. Water was obtained through a meter located at the District line at a pressure of about 55 lb. per sq. in. and raised about 40 lb. to the standpipe. Pressures around the town varied from 40 to 125 lb. per sq. in.

All materials were purchased through competitive bids, separate contracts being awarded for the welded steel standpipe and for the pumping equipment completely erected and installed. Alternate

FIG. 1.—WELDED STEEL STANDPIPE OF 250,000-GAL. CAPACITY.

proposals were received on unlined cast-iron pipe, steel pipe wrapped and lined, and cement-asbestos pipe. The award of the contract for "Transite" pipe resulted in a saving of about $10,000. The Federal contribution for materials amounted to approximately $21,500 and included the purchase of about 10,000 ft. of 12-in. and 10-in. cast-iron pipe.

The standpipe is of welded steel construction with a capacity of 250,000 gal., 81 ft. high and 23 ft. in diameter (Fig. 1). It stands on Cushings hill about one mile from the center of town and is placed at the proper elevation to give adequate fire protection to the School, Church, Town Hall and other nearby buildings. Basic Lead Chromate paint was used on the interior three coats and on the first coat of the exterior with two coats of aluminum paint as a finish.

The pumping station was constructed with W. P. A. labor and is of red brick construction with asphalt shingled roof (Fig. 2). It houses two electrically driven centrifugal pumping units of 250-g.p.m. capacity each, against a 100-ft. total dynamic head. Both of the units are driven by 15-hp., 220-volt, electric motors, but one unit has a gasoline engine attached for standby use. The units are automatically

FIG. 2.—LESURE AVENUE PUMPING STATION.

operated from pressure switches controlled by the water level in the standpipe (Figs. 3 and 4).

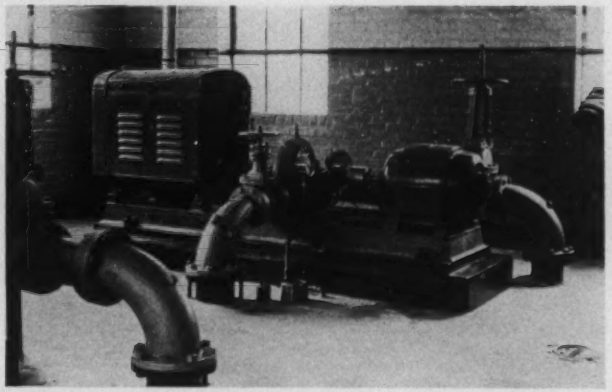


FIG. 3.—250-G.P.M. GASOLINE-ELECTRIC CENTRIFUGAL PUMPING UNIT.

The original computations for size of system were based on 175 houses being connected to the mains representing a conservative estimate of the number of takers for the first few years. Before the project was completed however, 255 services had been laid, connecting with practically 100% of all the houses along the pipe line. One of these services leading off the main street through private property has a length of 1880 ft. with an extension of 560 ft. Although this service pipe is of 2-in. copper pipe, no trouble from "blue water" has been experienced. Seventy-four fire hydrants were placed under this project so located as to be within 500 feet of all houses along the highway.

Before the completion of the first project in June 1940, numerous applications for extension of the Water District were received and it was evident that a second W. P. A. project was in order. A second meeting of the District authorized the issue of $40,000 in notes to lay about 21,000 ft. of additional pipe, to connect 76 new services, and to install 24 extra hydrants. The work on this new project is practically complete, and a third W. P. A. project has just been authorized to lay

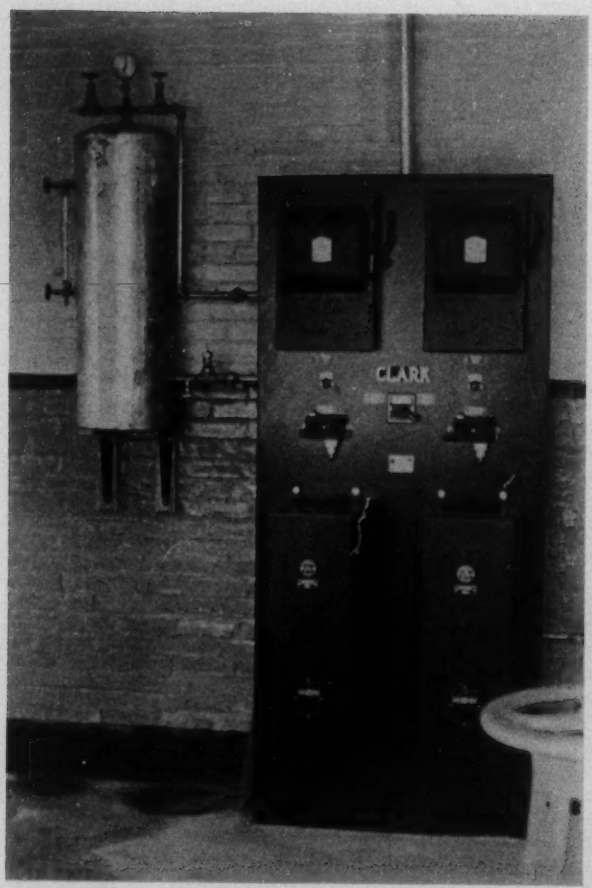


FIG. 4.—AUTOMATIC ELECTRICAL CONTROL EQUIPMENT.

about 10,000 ft. of pipe with 50 new services and 14 more hydrants to be financed by savings made on the two previous projects without issuing additional notes.

Summarizing the salient features of the Lunenburg Water Project, the District by issuing $124,000 in notes will have laid about 17 miles of pipe, constructed a standpipe and pumping station, erected 112 hydrants and connected 381 services. The minimum charge for water is $20.00 per year per service, and a hydrant rental of $40.00 each is charged to the Town of Lunenburg.

Can my town afford a water system? The answer is "Yes" as long as you can obtain a half million dollars worth of construction at an expenditure of $124,000. The limiting conditions appear to be that the town must be in good financial condition, Federal assistance must be available in the form of W. P. A. or similar agency disbursements, the houses served must not be too far apart averaging a spacing of not more than 400 ft., and the town must pay a reasonable hydrant rental of from $30.00 to $50.00 per hydrant or a similar allowance for its fire protection.

PROCEEDINGS

April Meeting

Brooklawn Country Club, Bridgeport, Connecticut
Thursday, April 24, 1941.

Vice-president Francis H. Kingsbury in the chair.

Secretary Gifford announced election to membership in the Association by the Executive Committee of the following:

Frederick Scott Gibbs, Sanitary Engineer for Wallace & Tiernan Co., Inc., Waban, Mass., and Robert Kaufman, Manager Greenwich Water Company, Greenwich, Conn.

A paper "Length of Service of Water Works Facilities—Knowledge or Guesswork?" was read by Reeves Newsom, Engineer-Consultant, New York, N. Y. Messrs. A. V. Ruggles and Theodore L. Bristol took part in the discussion.

A paper "The Saugatuck Development of the Bridgeport Hydraulic Company," was read by Samuel P. Senior, President, The Bridgeport Hydraulic Company. A motion picture in color, prepared by The Bridgeport Hydraulic Company, entitled "The Romance of a Glass of Water," was then shown.

At luncheon the Association was addressed by the Honorable Jasper McLevy, Mayor of the City of Bridgeport, Conn.

May Meeting

A Joint Meeting of the New England Water Works Association
and the New Hampshire Water Works Association

Nashua Country Club, Nashua, New Hampshire
Thurdsay, May 22, 1941.

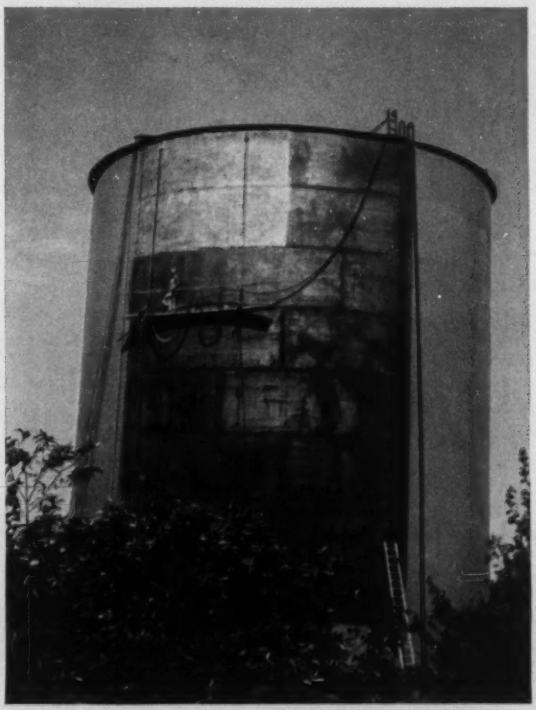
Percy A. Shaw, President of the New Hampshire Water Works Association, in the chair.

Secretary Gifford announced the election of the following members: Lyle S. Proffit, Superintendent, Water Department, Lunenburg, Mass., and Henry Chisholm, Superintendent of Public Works, Framingham, Mass.

The Association was welcomed to Nashua by William F. Sullivan, President of the Pennichuck Water Co., Nashua, N. H.

A paper, "Centrifugal Pumping Machinery," was read by E. D. Hinton, Engineer, Ingersoll-Rand Company. Percy A. Shaw and Arthur C. King took part in the discussion.

A paper, "Defense Training for Water Works Operators in Maine," was read by Harry U. Fuller, Chief Engineer, Portland Water District. Mr. Shaw discussed the paper.



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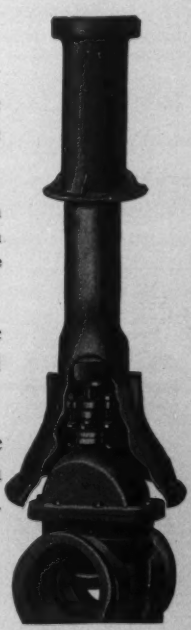

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BRANCH OFFICES NEW YORK PORTLAND, ORE. PHILADELPHIA
ATLANTA DALLAS CHICAGO SAN FRANCISCO LOS ANGELES

TRIDENT

Interchangeability

MODERN INTERCHANGEABLE PARTS
IN AN 1899 TRIDENT WATER METER

The Story of a Sound Investment

TWO essential facts set the Trident Water Meter apart from all others, as a profitable investment, and as a dependable source of Water Works revenue.

First—this water meter was first produced in a form so basically correct, so fundamentally right for its purpose, that its basic design has never been essentially changed. That is to say, later improvements have not made older Trident Meters obsolete.

Second—every essential improvement in Disc Water Meters made in the last 40 years is found in the modern Trident Meter today . . .

—and can be incorporated in Tridents made as long as 40 years ago, by means of modern Trident interchangeable parts.

Think! Even in 1899, there were such "modern" Trident pioneering features as the Breakable Bottom, the Thrust Roller and the Snap-Joint Disc Chamber, which protect accuracy and reduce maintenance cost in Trident Water Meters today.

At first, as the initial million Tridents were produced, improvements (such as heat proof renewable bushings) were minor . . . but the basic design of the Trident Meter remained unchanged. New models did not supplant older ones.

Then, as the Trident millions swiftly mounted, Neptune pioneered in new ways to improve and protect meter accuracy and prolong life—still without the necessity of changing the basically correct design of the meter, without making old models obsolete.

The year 1919 saw the introduction of such further technical developments as the Trident Oil-Enclosed Gear Train. Screwless Registers were perfected in 1921-24; the 3-part Disc in the latter years; the Protective Sand Ring two years later; then in 1939 came the Thrust Roller Bearing Plate . . . all readily inserted into old Trident Meters.

Remember one thing—all this time the basic Trident Meter design has remained unchanged. In other words, Trident Meters of yesterday can logically be made "better than new", by the insertion of the interchangeable parts of TODAY. Interchangeability was, and is, the basic principle and policy of Trident Meter design.

Today the Trident Meter remains, as it always has been, the best water meter on the market, if the testimony of more than 6 million made and sold (the great majority still in service) is a true criterion of value.

NEPTUNE METER COMPANY • 50 West 50th Street • NEW YORK CITY

Branch Offices in CHICAGO, SAN FRANCISCO, LOS ANGELES, PORTLAND, ORE., DENVER, DALLAS, KANSAS CITY, LOUISVILLE, ATLANTA, BOSTON

Neptune Meters, Ltd., 345 Sorauren Avenue, Toronto, Canada.

WATCH DOG
WATER METERS
. . . built to give
LASTING SATISFACTION
The Worthington-Gamon Meters of today represent an experience of 85 years . . . dating back to the invention of the piston meter, in 1855, by Henry R. Worthington.
With 3,650,000 meters now serving thousands of communities, this organization offers to municipalities and water companies a product whose accuracy and low maintenance requirements have proved it to be a sound investment.
Write for literature.
WORTHINGTON-GAMON METER COMPANY
General Offices: HARRISON, NEW JERSEY
WORTHINGTON-GAMON

WG1-2A

THE STANDARD FOR OVER 50 YEARS

Automatic Pressure Control Valves
Water Works Specialties

Altitude — Pressure Reducing — Surge
Relief and Combination Valves
Portable Fire Hydrants
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ROSS VALVE MFG. CO., Inc. Troy, N.Y.

Health and Protection First

Water delivered through dirty pipes
may be a MENACE.

Incrusted water pipes mean inefficiency
and loss of Fire Protection.

We Guarantee the Results of Our
Method of Cleaning.

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National Water Main Cleaning Co.
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Cement Lined Service Pipe

Cement lined pipe has eliminated corrosion and metal contamination for 60 years

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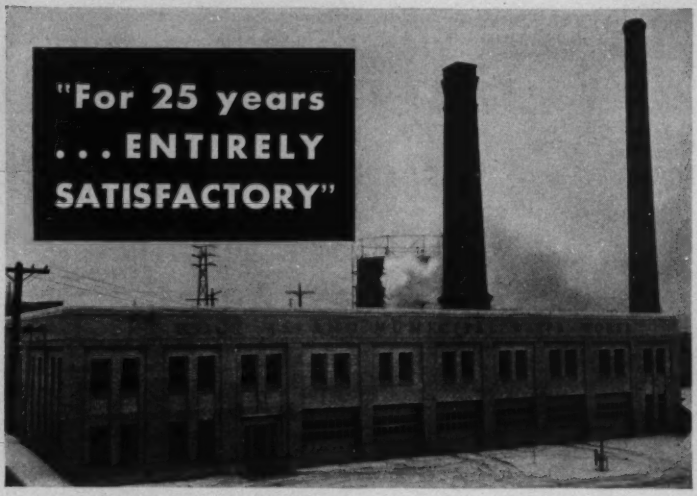


. . ROCK ISLAND Municipal Water Works, another Darling Valve user

To valve users everywhere, Darling's "dense-metal" construction means better machining, more accurate threading, and freedom from flaws or shrink — in short, *better performance*. From the smooth, uniform design . . . through careful foundry technique, accurate molding, proper gating and heading . . . every step in Darling production is directed toward the smooth, close structure known as "dense-metal". Get the benefit of this extra care . . . specify Darling's for your next installation!

DARLING VALVE & MANUFACTURING COMPANY
WILLIAMSPORT, PA.

Representatives in:
New York Philadelphia Houston Toledo
Pittsburgh Huntington, W. Va.
McPherson, Kan.

DARLING
DENSE-METAL
GATE VALVES and HYDRANTS

TIGHT JOINTS

REGARDLESS OF PIPE SIZE OR SOIL CONDITION

UNDER all conditions of service cast iron water mains can be laid with assurance of permanently "bottle-tight" joints if HYDRO-TITE is used as the jointing compound.

No matter how large or small the pipe nor how tough the going may be, HYDRO-TITE will prove the easiest and most economical to use.

HYDRO-TITE has a record of more than a quarter century without a failure anywhere.

A Symbol of Quality

A Dependable Self-Caulking Joint Compound

HYDRO-TITE

HYDRAULIC DEVELOPMENT CORPORATION

Main Sales Office: 50 Church Street, New York, N. Y.
General Offices and Works: West Medford Station, Boston, Mass.

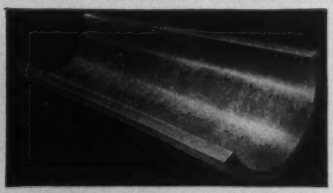
DELIVERY CAPACITY...

and how a non-tuberculating pipe helps keep it at a high level

Waterworks engineers generally agree that a water main, even though it remain structurally intact, may completely outlive its economic usefulness over a period of time. This point is reached when the pipe will no longer carry, economically, a sufficient volume of water to meet demand. Such a condition may arise from characteristics inherent in the pipe itself, from increased demand or from a combination of these two causes. Thus, in designing water systems, the engineer is faced with the responsibility of anticipating and solving these problems.

A Practical Solution

To compensate for expected reductions in carrying capacity, specifications often call for larger sizes of pipe than would otherwise be necessary. However, there is another, less expensive solution. In J-M Transite Pipe, engineers find basic advantages that permit them to specify the most economical pipe sizes with full assurance that delivery capacity will not be reduced by tuberculation, the internal corrosion which, even when present in relatively small degree, greatly reduces the carrying capacity of ordinary pipe.



Transite Pipe provides an unusually smooth interior surface that offers minimum resistance to the flow of water.

Flow Coefficient C-140

Transite Pipe is an asbestos-cement product. During manufacture, it is built up on a polished steel mandrel that imparts unusual smoothness to the interior wall. Its flow coefficient is conservatively estimated at C-140. This means that Transite water lines start with an exceptionally high carrying capacity.

Tests conducted by the Pitometer Company on a typical Transite water line revealed a flow coefficient of C-145 in service.

Economies due to Freedom from Tuberculation

Transite's freedom from tuberculation means that pumping costs stay low, for pressures need not be increased to compensate for steadily decreasing flow coefficient and resulting loss of head in the line. Not only does this advantage provide worth-while operating economies, but in many cases it also permits the use of smaller diameter pipe with resultant savings in all the items that make up the original cost of completed water lines.

In addition to sustained high carrying capacity, Transite Pipe has other advantages which contribute to more economical water transportation. In hundreds of American municipalities, it has also brought soil corrosion, joint leakage and electrolysis under control. And in every case low installation costs are assured because of the ease and speed with which Transite's Simplex Couplings are assembled.

NOTE TO ENGINEERS: A much more complete discussion of the relationship between tuberculation, carrying capacity and pumping costs is given in the Transite Pressure Pipe brochure, TR-IIA. A copy of this book will be sent on request. Johns-Manville, 22 E. 40th St., N. Y. C.

Johns-Manville TRANSITE PIPE

For efficient, economical water and sewer lines

The Heart of the Empire STREEEMLINE METER

THE TIME PROVEN, BALANCED, OSCILLATING PISTON

THERE is but one moving part within the measuring chamber of the Streeemline Meter, the famous EMPIRE Balanced Piston. The perfect balance in this piston is attained by the location of the web at the vertical center of the piston. The pressure on both sides of the web is thus equalized resulting in a piston that practically glides between the top and bottom half of the chamber with minimum friction and wear.

On the bottom circumference of the oscillating piston is located a hard rubber roller bearing called the Piston Pilot that operates in a slot cut in the lower half of the measuring chamber on the inlet side. The Piston Pilot, besides acting as a guide to control the movement of the piston, eliminates all contact between the piston and the division plate. Its rolling action prevents friction, thereby providing greater sensitivity, increased accuracy at low flow rates, and assures sustained, quiet operation.

The balanced oscillating piston is only one of the many features of the EMPIRE Streeemline. This meter is the culmination of 71 years of measurement experience. Its construction embodies the indicated preferences of many water works men. It provides the unequalled performance characteristics of the time proven oscillating piston design, plus improvements and refinements resulting from present day knowledge and engineering advances.

All the features of the EMPIRE Streeemline Meter are described in detail in a new Catalog N-307. Write for a copy or arrange to have a Pittsburgh-National representative call.

PITTSBURGH EQUITABLE METER COMPANY

NEW YORK BROOKLYN SEE MOINES ATLANTA DALLAS TULSA CHICAGO BOSTON

MERCO NORDSTROM VALVE COMPANY
Main Offices, Pittsburgh, Pa.
NATIONAL METER DIVISION, Brooklyn, N. Y.

KANSAS CITY PHILADELPHIA SAN FRANCISCO LOS ANGELES SEATTLE PORTLAND COLUMBUS BUFFALO

Pittsburgh-National Meters

THE MOST COMPLETE LINE OF WATER METERS IN THE WORLD

K & M *"Century"* PIPE TUNNELS TWO MOUNTAINS WITH THE FAMOUS PENNSYLVANIA TURNPIKE

Where the Pennsylvania Turnpike tunnels under Kittatinny Mountain, "Century" Pipe was installed to provide water for flushing the roadway.

The Pennsylvania Turnpike dives straight through mountains . . . and under two of them it takes with it 9,800 feet of Keasbey & Mattison "Century" Asbestos-Cement Pressure Pipe, to provide water for flushing the roadway.

The pipe had to be good. You don't install anything under a mountain except what you *know* will endure. "Century" Pipe was selected because of its permanence. It is not only strong, but corrosion-proof and tuberculation-proof. Its interior walls stay smooth.

"Century" Flexible Couplings are easily installed and are absolutely leak-proof. With the light weight and handling ease of the pipe itself, they cut laying cost to the minimum. Write Dept. 1017 for free booklet, "Mains without Maintenance."

The Pennsylvania Turnpike project broke speed records . . . and "Century" Pipe played its part. Notice how easily two men handle a 13-foot length. Light weight and handling ease cut installation cost.

Mr. Dave Agnew, Superintendent of Bates and Rogers Construction Corp., Chicago, who says, "This job called for durable, dependable materials easy to work with. 'Century' Pipe fits those requirements like a glove."

KEASBEY & MATTISON
COMPANY, AMBLER, PENNSYLVANIA

The FIRE HYDRANT with Extra values

Extra Convenience . . . .

The Kennedy SAFETOP can be lengthened for change in street grade without excavating or shutting off the water supply, its nozzles can quickly be made to face exactly any desired direction, and all operating parts are readily accessible.

Extra Economy . . . .

Repairs to a Kennedy SAFETOP broken in a smashing collision require only a standpipe breaking ring and a stem coupling costing $6.00. These parts can be installed in less than half an hour by one man, without excavating or shutting off the water supply. Other SAFETOP economy factors are the long-wearing valve and gasket materials, and the sturdy operating mechanism.

Extra Responsiveness . . . .

Large standpipes and inlet valves, carefully shaped entrance elbow, bell-shaped nozzles, and absence of obstructions through the hydrant combine to minimize friction loss and to assure practically full main pressure at the nozzles. In addition, the simple straight-line mechanism assures easy operation.

Extra Safety . . . .

When struck by an impact which *no* hydrant could withstand, the Safety Breakable Section of the Kennedy SAFETOP parts cleanly, and no flooding can occur. Occupants of the colliding vehicle are protected from likelihood of severe personal injury, and the neighborhood is protected from damage by water or lengthy delay in returning the hydrant to service.

Send for Bulletins

The Kennedy Valve Mfg. Co., Elmira, N. Y.

KENNEDY
SAFETOP FIRE HYDRANT
REG. U.S. PAT. OFF.

EDDY

Valves and Hydrants

Distinctive Features of the EDDY Valves

ONLY three parts are moved by the Stem—the ball and the two gates. Each gate is hung from one of the trunnions on the ball. A convex surface at the base of each trunnion fits a concave surface on the back of the gate. This allows the gates to adjust themselves properly to their seats. . . . The gates, being free to revolve on the trunnions, do not always seat in the same position. Gates are center bearing and adjustable. They are forced to their seats with equal pressure at all points. There are two hooks on the ball on sizes 4-inch and larger, which loosely engage with the gates.

Distinctive Features of Our New Swivel Top Hydrants

THIS new hydrant has all the advantages of the popular EDDY fire hydrant, plus several additional features. . . . Nozzles are in a short flanged section of the standpipe, which may be unbolted and turned to different positions. . . . Should the standpipe be broken, only the casting below the swivel head need be replaced. . . . To raise the hydrant to conform to a new grading, simply insert a flanged extension piece below the swivelhead. No digging. To add a steamer nozzle at any time, it is only necessary to replace the swivel head with one having a steamer nozzle.

Besides valves and hydrants to meet the most exacting demands of waterworks service, the complete EDDY Line includes: valves for sewage disposal works; valves for steam, gas and oil; check valves, foot valves, plug valves, shear gates, indicator posts, etc. EDDY valves are made in three classes: Iron Body Bronze Mounted, All Bronze, and All Iron Valves.

Eddy Valve Co.

WATERFORD NEW YORK

LEAD PIPE

Used by leading WATER WORKS for over 50 years. If proper weight is used, combined with good workmanship, your FINAL COST is assured, as REPAIRS are unnecessary. LEAD PIPE IS sufficiently ductile to conform to any sagging condition. There is no RUSTING of pipe or joints; no DISCOLORATION of water.

HIGHEST QUALITY SOFT OMAHA PIG LEAD

LEAD WOOL

Every atom pure lead

For calking pipe joints under the most difficult conditions.

RED LEAD

For rust-proofing tanks and pipes.

Write for Water Works specifications.

PURE BLOCK TIN PIPE **TIN-LINED LEAD PIPE**

DUTCH BOY WHITE LEAD

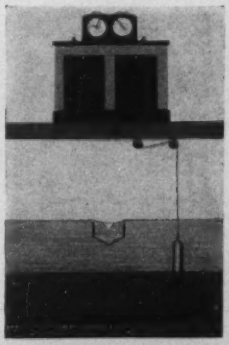
SOLDER

National-Boston Lead Co.

800 Albany Street **Boston, Mass.**

BUILDERS SAND EXPANSION GAUGE

This is the latest addition to the Builders line of water works and sewage metering and controlling equipment. It enables the operator to control both the *extent* and the *rate* of sand expansion in the filter bed. It operates independently of changes in wash water flow, pressure, or temperature. Indicators and Recorders can be furnished in standard attractive designs for table or floor stand mounting.



Kennison Nozzle—for open flow

Simple, accurate and trouble-free, Kennison Nozzles have been widely installed for measuring raw sewage, sludge or effluent.

Descriptive bulletins showing Builders Meters, Gauges, and Controllers free on request, write

BUILDERS - PROVIDENCE, INC.

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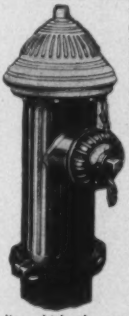
THE A. P. SMITH MFG. CO.,
East Orange, New Jersey



Manufacturers of high-grade specialties in connection with water and gas department work, consisting of—
Machines for inserting valves under pressure, and for making large right angle connections to mains under pressure, and for inserting house service connections under pressure.
Also makers of—
Pipe Cutting Machines, each size being adjustable for use on three different sizes of pipe.
High-grade water works supplies, such as Hydrants, Valves, Meters, Corporation Cocks, etc.
We also make a new type of hydrant which has a breakable joint at the ground line which when struck with an impediment will break at a specified point and only a coupling has to be used to repair the hydrant, which will be back in service in twenty (20) minutes.
"HYDRANT WITH BREAKABLE COUPLING"

SMITH
WATER WORKS
SPECIALTIES & SUPPLIES
THE A.P.SMITH MFG. CO. East Orange, N.J

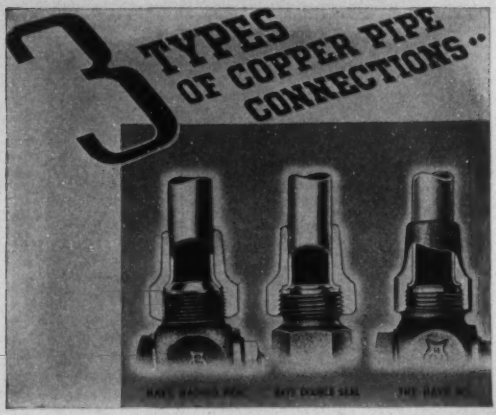
THE FORD METER MITTEN
WILL SAVE FROZEN METERS

Made for meters up to 1" in size.

Simple and easy to install.

The Meter Mitten is a convenient and effective insulation jacket for basement meters. Conserves warmth in meter and water and prevents many meters from freezing. Costs but a fraction of the expense of a frozen meter. Write for further information.

The Ford Meter Box Co.
WABASH, INDIANA



Whatever tapping method or type of corporation cock your city uses, Hays can take off with copper pipe to the curb or to the meter. Hundreds of water works departments have found Hays permanent copper services readily adaptable to their needs at little or no extra cost over short life methods.

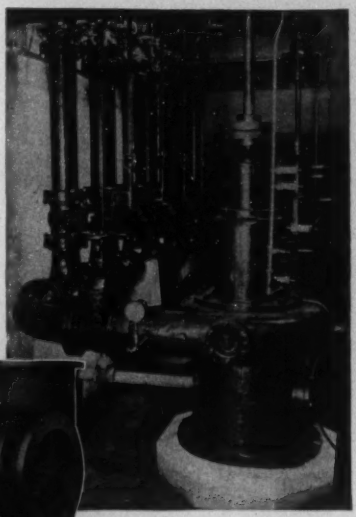
HAYS MFG. COMPANY Erie, Pa.

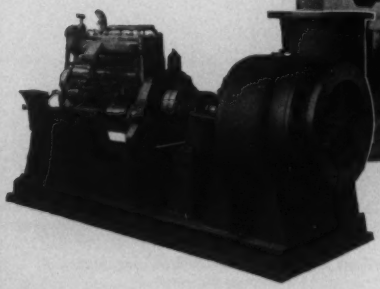
SINCE 1869

HAYS

High Efficiency and Dependability

in water supply and sewage services





Morris Centrifugal Pumps have made notable records in municipal pumping services . . . notable for low power consumption, trouble-free operation, low maintenance costs, and long life.

Write for bulletins describing Morris High-Efficiency Water Supply Pumps and Morris Non-Clogging Sewage Pumps.

STARKWEATHER ENGINEERING CO., Inc.

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CENTRIFUGAL PUMPS

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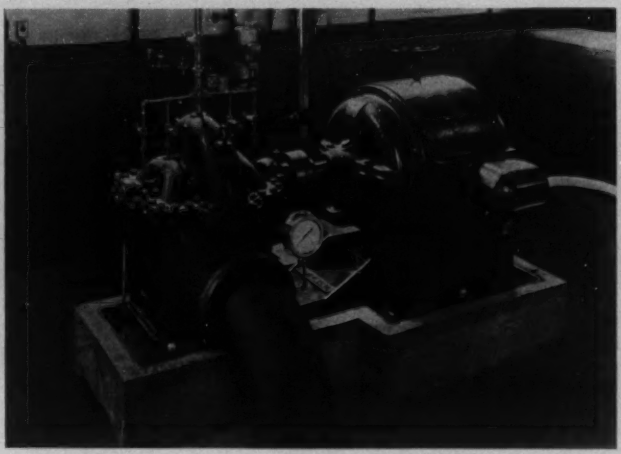
CENTRIFUGAL PUMPS FOR NATIONAL DEFENSE



200 H.P. Motor Driven Pumping Unit in the Grove Pond Pumping Station at Fort Devens, Ayer, Massachusetts

The photograph shows one of the duplicate units recently installed in the main station at Fort Devens. Each pump delivers 2100 gallons per minute against a total dynamic head of 280 feet including a suction lift.

These units represent part of a single contract which included complete erection together with two units in another station with all electric wiring, switchgear and priming equipment. Our contract was based on specifications prepared by F. A. Barbour, Supervising Engineer.

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80 Federal Street **Phone LIBerty 5993-5994** **Boston, Mass.**

New England Representatives for

DE LAVAL STEAM TURBINE COMPANY
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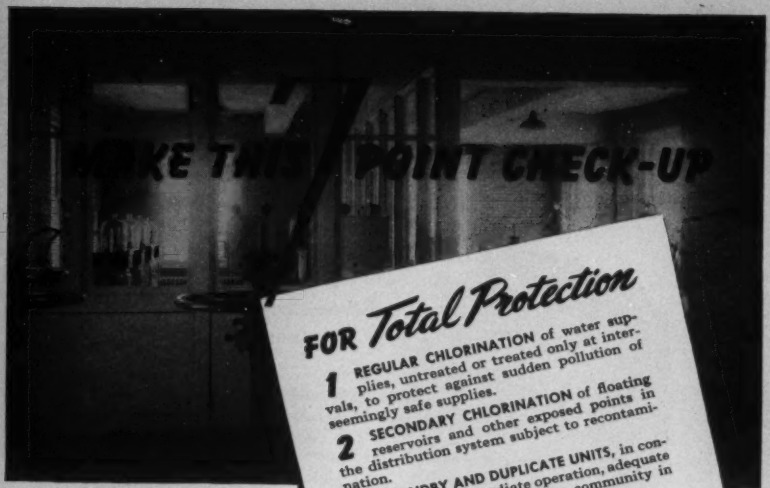


FOR *Total Protection*

1 **REGULAR CHLORINATION** of water supplies, untreated or treated only at intervals, to protect against sudden pollution of seemingly safe supplies.

2 **SECONDARY CHLORINATION** of floating reservoirs and other exposed points in the distribution system subject to recontamination.

3 **STANDBY AND DUPLICATE UNITS,** in condition for immediate operation, adequate to serve the full needs of the community in any emergency.

4 **FULLY MODERN EQUIPMENT** with the flexibility and dependability necessary to meet increased demands or emergency conditions with economy of service personnel.

5 **SPECIAL TREATMENT METHODS** (such as ammonia-chlorine) to maintain chlorine residuals throughout the system.

6 **EMERGENCY CHLORINATING EQUIPMENT.** Portable or mobile units, preferably self-powered, of a capacity adequate to cope with any local emergency.

7 **PERMANENT RECORDS** of chlorine dosage, kept continuously as a check on performance, on operators and equipment.

HOME DEFENSE STARTS WITH HEALTH DEFENSE

In planning for total protection of water supplies against every sort of emergency —natural or man-made, accidental or deliberate—your primary defense weapon is *chlorination.* Only chlorination can act to overcome contamination even *before* bacteriological examinations reveal its presence.

Here, in the W&T *"Safety Seven,"* are the points to check and consider to secure the full affectiveness of chlorination. All are sound practice at any time, and your investment now in adequate chlorinating equipment makes preparedness certain.

Whatever your water-treatment problem . . . extra safety . . . taste and odor control . . . low-cost plant operation . . . you'll find a W&T Representative at your service with experienced aid. Just call or write. SA114

HEALTH DEFENSE STARTS WITH STERILIZED WATER

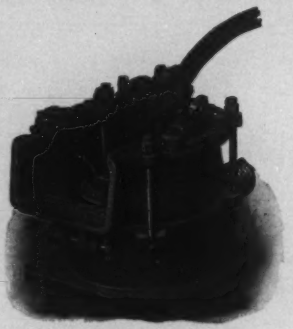
"The Only Safe Water is a Sterilized Water"

WALLACE & TIERNAN CO., Inc.

MANUFACTURERS OF CHLORINE AND AMMONIA CONTROL APPARATUS

BELLEVILLE, NEW JERSEY . . . REPRESENTATIVES IN PRINCIPAL CITIES

EDSON
PUMPS and ACCESSORIES



Hand Pumps – Power Pumps
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Distributors for
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Universal Water Leak Detector

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ALSO
Warren Pipe Co. of Mass., Inc.

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Manufacturers of

CAST IRON PIPE

Flanged Pipe ***Flexible Joint Pipe***
Bell and Spigot Pipe
Special Castings ***Short Body B. & S. Specials***
Warren (W) Spun Centrifugally Cast Iron Pipe

WORKS: PHILLIPSBURG, N. J. and EVERETT, MASS.

Large Stock Enables Us to Make Prompt Shipments

For an Extremely Smooth Pipe Interior
SPECIFY

SPUN **BITUMASTIC** LINING

This Will Assure You of
NO LOSS IN CAPACITY
NO TUBERCULATION OR INCRUSTATION
NO EFFECT ON WATER QUALITY

Recent Repeat Flow Test Shows High Williams Hazen Coefficient Unchanged After EIGHT YEARS

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CONE VALVES
(Patented Hydraulic Operating Device)

Used for
- ALTITUDE CONTROL
- PRESSURE RELIEF
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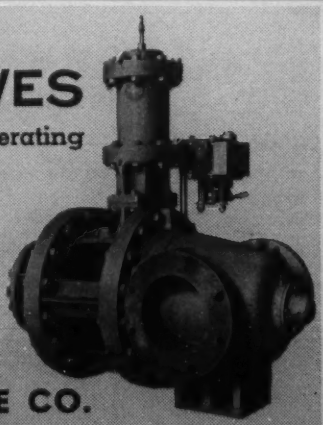
- AUTOMATIC CHECK



RENSSELAER VALVE CO.
TROY, NEW YORK

HERBERT B. CROWELL
Sales Manager, New England Branch
3 WASHINGTON SQUARE HAVERHILL, MASSACHUSETTS

HOW COMPLETE
ARE YOUR REFERENCE FILES?

EXTENSIVE LITERATURE ON TURBINE, POWER, SPLIT CASE AND BUILTOGETHER CENTRIFUGAL PUMPS IS AVAILABLE TO YOU — TOGETHER WITH REMARKABLY DETAILED BULLETINS ON THE MOTORS AND DIESELS THAT DRIVE THESE PUMPS.

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Write to

FAIRBANKS, MORSE & CO.

178 Atlantic Avenue **Boston, Mass.**

Soon to be started: Installation of still another Fairbanks-Morse Diesel-driven Pump, this time for the Exeter, N. H., Water Company.

This, concisely put, is what you get when you use Tegul-MINERALEAD in jointing your Bell and Spigot Pipe ■ The ingot form is impervious to rain and flood ■ This compound works with pipe of any diameter, isn't bothered by terrain conditions and has the stamina to withstand far greater vibration and punishment than it will ever meet in service ■ The cost will surprise you pleasantly ■ For complete information, write the ATLAS MINERAL Products Company of Pa. Mertztown, Pennsylvania.

Tegul- MINERALEAD

For Jointing BELL & SPIGOT PIPE

DONALDSON IRON CO.

MANUFACTURERS

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Special Castings for Water and Gas

Also Flange Pipe and Fittings

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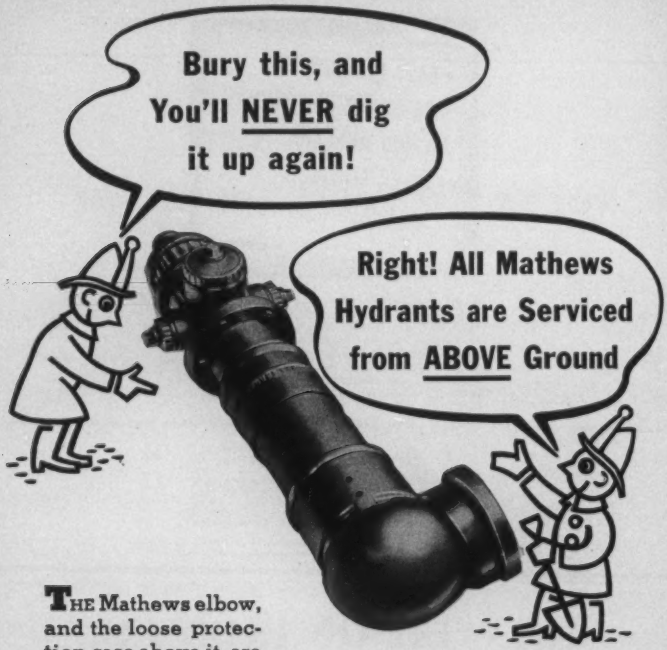

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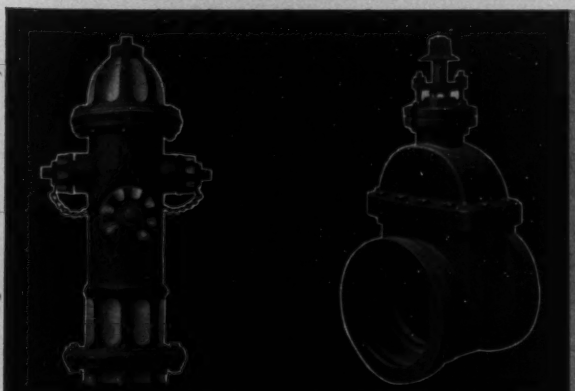

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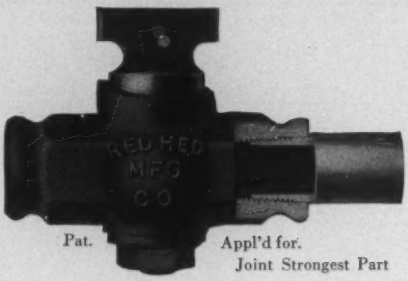

Fig. 220

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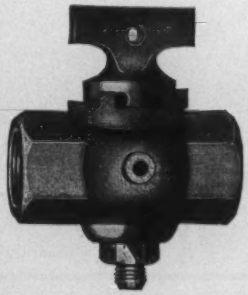

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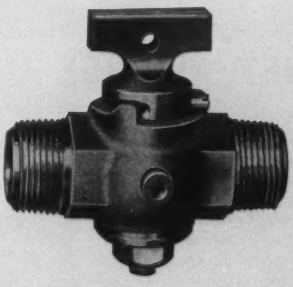

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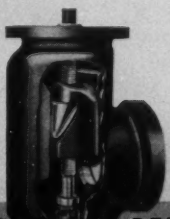

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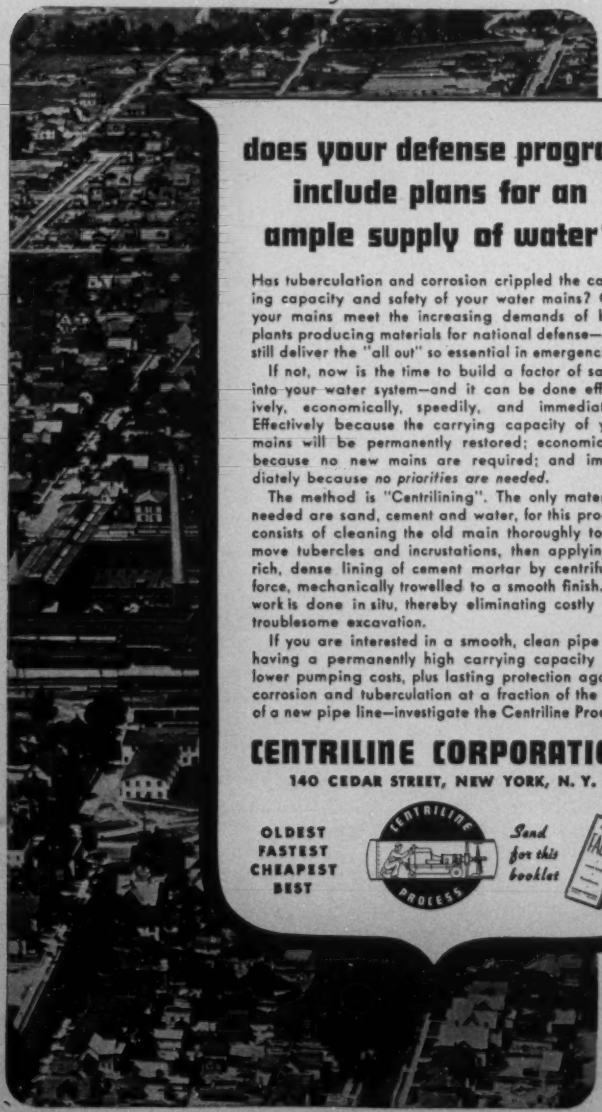

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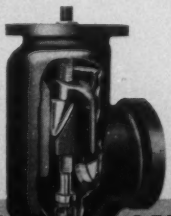

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